CRANFIELD UNIVERSITY

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Vertical Flow Wetlands for Tertiary Wastewater Treatment

School of Energy, Environment and Agrifood

EngD
Academic Year: 2016 - 2017

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This thesis is submitted in partial fulfilment of the requirements for the degree of EngD

Abstract

Due to increasing focus on improving water quality within surface waters, it is anticipated that stringent ammonia discharge consents will be introduced to small wastewater treatment works in the coming years, with potential discharge consents of as low as 1mgNH_4-N/L. This is likely to require upgrading of secondary treatment works to include a polishing treatment stage. Vertical flow wetlands (VFWs) are aerobic treatment processes, making them the ideal solution for achieving nitrification to within the proposed discharge consent, however current uses are limited to treatment of raw water and primary effluents. The potential for VFWs under tertiary application has not previously been identified. This study aims at addressing this knowledge gap by determining performance capabilities and hydraulic behaviours at both full and pilot scale, and defining the optimal operational strategies in terms of hydraulic loading, dosing frequency and resting periods using pilot plant trials. Findings from the study have shown VFWs to achieve effluent ammonia concentrations of as low as 0.002mgNH_4-N/L from influent concentrations of up to 7.4mgNH_4-N/L, with almost complete nitrification observed in most cases. Additional onsite sampling provided a performance comparison against existing tertiary treatments, showing potential for the VFWs to outperform in terms of solids organics and nutrient removal. Pilot plant operational trials revealed application of prolonged resting periods and frequency of daily dosing to have no significant impact on either the hydraulic stability or treatment performance for VFWs in tertiary application. Pilot plant trials determined an initial stabilisation period of between 2 to 3 years is required during the start up of a VFW system, with hydraulic loading rates being increased gradually over time to avoid occurrence of clogging. An economic assessment determined the feasibility of tertiary VFWs to be comparable to existing conventional tertiary treatments.

Keywords:

Nitrification, Nutrient Removal, Hydraulic Loading Rate, Dosing Frequency, Resting Periods.

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LIST OF EQUATIONS

LIST OF ABBREVIATIONS

AHFW	Aerated Horizontal Flow Wetland
ANOVA	Analysis of Variance
AOB	Ammonia Oxidising Bacteria
APHA	American Public Health Association
As	Arsenic
ASP	Activated Sludge Process
atm	Atmospheric Pressure
BOD	Biochemical Oxygen Demand
Ca	Calcium
CAPEX	Capital Expenditure
Cd	Cadmium
CEMAGREF	Centre d'etude du Machinisme Agicole du Génie Rural des Eaux et Forêts
cm	centimetres
COD	Chemical Oxygen Demand
Cr	Chromium
Cu	Copper
DDF	Daily Dosing Frequency
DEFRA	Department for Environmental, Food and Rural Affairs
DF	Dosing Frequency
DNA	Deoxyribo Nucleic Acid
DO	Dissolved Oxygen
DWF	Dry Weather Flow
EPS	Extracellular Polymeric Substance
FAAS	Flame Atomic Adsorption Spectrometry
Fe	Iron
FE	Final Effluent
g	grams
GRP	Glass Reinforced Plastic
HC	Hydraulic Conductivity
HCl	Hydrochloric Acid
HCO_3	Bicarbonate Hydrogen Carbonate

HF	Horizontal Flow
HFW	Horizontal Flow Wetland
Hg	Mercury
HLR	Hydraulic Loading Rate
HM	Heavy Metals
HRT	Hydraulic Retention Time
ICP-MS	Inductively Coupled Plasma – Mass Spectrometry
IRSTEA	National Research Institute of Science and Technology for Environment and Agriculture
KW	Kilowatt
KWh	Kilowatt hours
L	Litre
LOD	Limit of Detection
m	metres
M	Molar
MBR	Membrane bio-reactor
M&E	Mechanical and Electrical
Mg	Magnesium
mg	miligrams
mm	millimetres
Mn	Manganese
n	Number of Samples
N	Nitrogen
NH_4-N	Ammonium-Nitrogen
Ni	Nickel
NO_2-N	Nitrite-Nitrogen
NO_3-N	Nitrate-Nitrogen
NSAF	Nitrifying Submerged Aeration Filter
OLR	Organic Loading Rate
OM	Organic Matter
OPEX	Operational Expenditure
ORP	Oxidation-Reduction Potential
P	Phosphorus
Pb	Lead

PE	Population Equivalent
PPCP	Pharmaceuticals and Personal Care Products
PPM	Parts Per Million
PVC	Polyvinyl Chloride
RBC	Rotating Biological Contactor
RP	Resting Periods
SF	Sand Filter
SLR	Solids Loading Rate
TF	Trickling Filter
TKN	Total Kjeldahl Nitrogen
TP	Total Phosphorus
UK	United Kingdom
UKWIR	UK Water Industry Research
USEPA	United States Environmental Protection Agency
VF	Vertical Flow
VFW	Vertical Flow Wetland
VFW(s)	Siphoned Vertical Flow Wetland
WLC	Whole Life Cost
WWTW	Waste Water Treatment Works
Zn	Zinc
µg	Micrograms

Chapter 1

Introduction

Introduction

1.1 Background

Nutrient levels within watercourses have a strong influence over the health of aquatic ecosystems and, when present at excessive levels, can result in eutrophication. Discharges from wastewater treatment works are considered a significant source of such nutrients, particularly ammonia and phosphorus. As part of ongoing activities to manage the overall health of ecosystems, discharge consents from sewage works are expected to become more stringent. Previous control measures have focussed on medium and large sewage works but more attention is now being directed towards small works (sub 2000 pe) as part of the water framework directive (Directive 2000/60/EC), as small works account for around 75% of all treatment works in the UK. To illustrate, in the case of ammonia, small works with no previous numeric discharge consent are expecting the introduction of discharge consents of 4mgNH_4-N/L with some sites expecting to achieve ammonia discharge consents of less than 1mgNH_4-N/L, depending on the available dilution and ecological status of the receiving water course.

Traditional small works utilise relatively passive biological processes, such as trickling filters or rotating biological contactors, whose adaptation to meet such tightening discharge consents will be challenging. Accordingly, consideration is being directed towards the role of tertiary treatment systems in supporting additional ammonia removal. Critically, ammonia removal is driven through an aerobic biological transformation pathway such that sufficient oxygen needs to be available to meet both the consumed demand for ammonia degradation and respiration, normally ensured by maintaining a residual dissolved oxygen concentration in the water of above 1mg/L, but optimally between 3-4mg/L (Stefanakis, et al., 2014).

Traditional tertiary treatment technologies such as depth filters and constructed wetlands were originally designed to remove additional solids (and associated particulate organics) and hence had no need to ensure aerobic conditions were present. Consequently, such systems do not normally remove ammonia and so adaptation is required. In the small works context, this is perhaps most apparent in the case of constructed wetlands. Current preference is to use horizontal sub surface flow wetlands that remain permanently saturated, creating predominately anaerobic and anoxic environments. Recent developments have seen adaptation through the inclusion of coarse bubble aeration systems which then render the wetland aerobic enabling nitrification and the ability to meet very tight ammonia discharge consents (Butterworth et al., 2013). However, the inclusion of forced aeration deviates from the underlying passive philosophy of wetland technology providing a space for alternative systems to be considered. The most apparent alternative is that of vertical flow wetlands (VFW) which provide passive aeration and are commonly applied for whole or secondary treatment for private discharges (Weedon, 2010) and for municipal treatment in other countries such as France (Molle et al., 2005; Paing and Voisin, 2005). However, the application of VFWs remain uncommon for tertiary upgrade of sewage treatment (Besançon et al., 2017).

Vertical flow wetlands are fed influent wastewater via a network of above ground pipework providing an evenly distributed flow over the entire surface of the bed, limiting the potential development of preferential pathways. The body of these wetlands typically consist of a deep main treatment media layer of sand at the top of the bed, a transition layer of pea gravel and a drainage layer of large gravel or cobles at the bottom of the bed, with the treated effluent being collected from within this drainage layer. The media type and configuration promotes physical entrapment of solid material and provides a large surface area for biofilm adhesion and establishment, enhancing biological wastewater treatment through increased water to biofilm contact time. Wetlands are often planted with a reed species, typically *Phragmites australis*, of which the roots are thought to have a positive action on performance due to the provision of additional surface area for biofilm adhesion, oxygenated root zone area and increased hydraulic conductivity with root movement, in addition to direct nutrient uptake (Stefanakis et al., 2014; Vymazal et al., 1998). Vertical flow wetlands for wastewater treatment have shown that initial process stabilisation can take up to 4 years, during which the hydraulic response of the systems alters (Chazarenc and Merlin, 2005; Vanier and Dahab, 2001). As such it is expected that hydraulic acceptability will increase with continued operation.

Vertical flow wetlands generate aerobic environments by application of intermittent batch loading such that the biofilm is cyclically exposed to feed and oxygen (Figure 1-1). The specific balance of feed and rest cycles is set in response to the limiting component. Current application of VFWs is predominately for secondary treatment where the relatively high pollution loads render the systems oxygen limited and hence operate with prolonged rest cycles to ensure sufficient oxygen can be transferred into the biofilms. Conversely we posit that adaptation to tertiary applications switches the rate limiting component to that of load due to the relatively low pollution concentration that exist in secondary effluents (Table 1.1). Consequently, the operational strategies applied to current secondary treatment VFWs are not likely to be applicable for tertiary application, therefore there is a need to identify the extent of the VFW treatment mechanisms involved within tertiary application and determine how these differ from secondary application so that appropriate design and operational strategies can be defined.

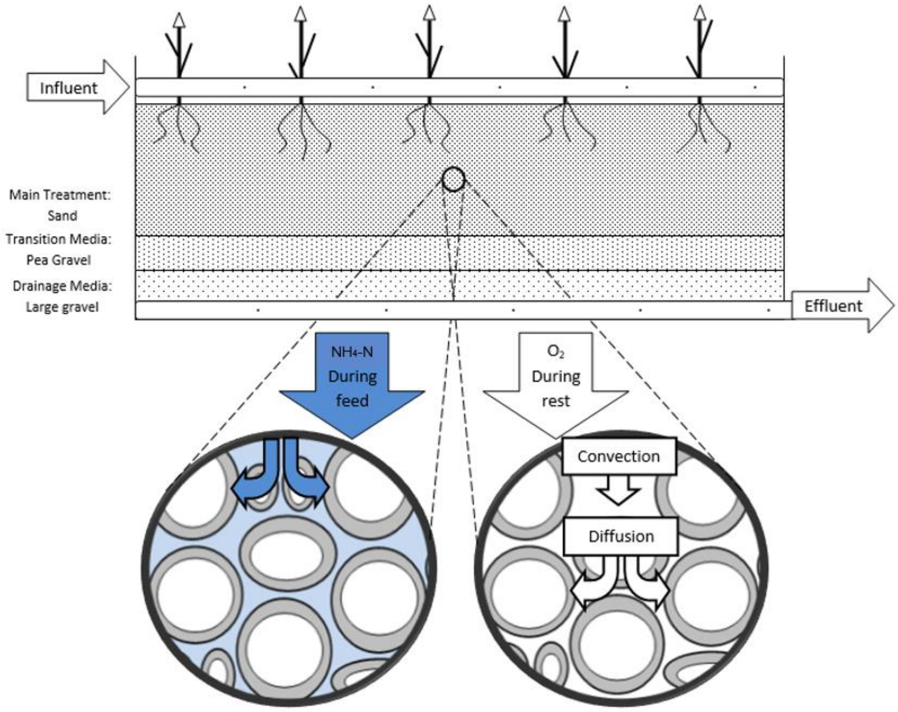

Figure 1-1 Schematic of a VFW, illustrating the transfer routes of ammonium-nitrogen and oxygen into the media voids and biofilm during the alternating feed and rest cycle.

Table 1-1 Comparison of influential factors regarding treatment in whole/secondary and tertiary VFWs.

Influential Factor	Whole/ Secondary Treatment	Tertiary Treatment
Pollutant load	Higher	Lower
HLR	Lower	Higher
Biomass	Higher	Lower
Oxygen demand	Higher	Lower
Treatment limitations	Low oxygen (nitrification failure)	Low pollutant load (Possible hydraulic overload)
Treatment enhancement	Increase oxygen transfer (long rest periods)	Increase pollutant load and biofilm to water contact time (short rest periods)

1.2 Project Aims and Objectives

The aim of the thesis is to understand the impact of changing design and operating parameters on the efficacy of vertical flow wetlands, and to establish the feasibility of the technology as a tertiary treatment process.

To achieve this aim, the following objectives have been identified:

1. Review existing design and operational procedures to understand the potential impacts on tertiary treatment.
2. Determine the influence of loading rate on the performance of tertiary vertical flow wetlands.
3. Determine the influence of dosing frequency on the performance of tertiary vertical flow wetlands.
4. Assess the effectiveness of resting periods on clogging control and prevention in tertiary vertical flow wetlands.
5. Determine the impact of design and operational choices on the economic and environmental suitability of vertical flow wetlands for tertiary treatment.

1.3 Thesis Structure

The thesis is presented as chapters formatted in the style of journal papers for publication. All papers were written by the first author, Nicole Jenkins, and edited by Professor Bruce Jefferson and Dr Gaby Dotro. All experimental work was carried out by Nicole Jenkins, either onsite at the selected Severn Trent Water sewage treatment works, or at Severn Trent Water and Cranfield laboratories. The link between the different chapters of the thesis and the objectives addressed in each chapter are outlined in Table 1-2 and illustrated in figure 1-2. The thesis addressed the different packages of work with the following content:

Chapter 2: Vertical flow constructed wetlands for municipal wastewater treatment: A review to assess the potential for use as a tertiary treatment process.

To ascertain the current status of wastewater treatment with vertical flow wetlands a literature review was conducted, revealing applications including whole, secondary and advanced treatment of wastewaters. The review analyses and compares the impact of VFW designs on the treatment performance efficiency through the identification and understanding of the pollutant removal mechanisms involved. The review concludes with recommended design considerations for tertiary application, which were applied to the VFW design for the pilot plant trials.

Chapter 3: Low energy tertiary treatment with vertical flow wetlands: a UK case study

A case study was conducted to assess the performance of a mature, gravel based, full scale vertical flow wetland utilised as a tertiary treatment process on a small works. The aim of the study was to determine how the current tertiary vertical flow wetland design and operation influenced the removal mechanisms involved in pollutant removal. This was achieved by measurement of solids, organics and nutrient removal and hydraulic behaviour to establish a performance profile. The study concludes with recommendations for design improvement to enhance treatment capacity at tertiary scale.

Chapter 4: Impact of loading rate on the operation and performance efficiency of pilot scale tertiary vertical flow wetlands

This study assessed the effect of the hydraulic loading rate on the performance efficiency and determined the hydraulic limitations through the use of eight pilot plant VFWs over two separate experimental phases. The aim across both experimental phases was to determine how the hydraulic loading rate influenced the hydraulic effectiveness and treatment performance of an unplanted immature wetland. The trials were expected to represent a conservative assessment of hydraulic loading as VFWs are known to require between 1-3 years for the beds to mature and performance to stabilise, especially with respect to hydraulic throughput.

Chapter 5: Impact of dosing frequency on the nitrification potential within tertiary vertical flow wetlands

This chapter saw the continuation of the Tertiary VFW operation optimisation study by assessing the impact of dosing frequency on the overall performance efficiency, with particular attention on the nitrification capacity. This was conducted on six pilot plant VFWs using a variety of daily dosing frequencies to determine the impact of contact ratio between air and biofilm during resting periods and between water and biofilm during feeding periods on the performance efficiency of the system. A performance comparison of implementing the technology against conventional treatments was carried out to determine the viability of the system within tertiary application.

Chapter 6: Impact of resting periods on the hydraulic behaviour and performance potential within tertiary vertical flow wetlands

In the final period of pilot testing, the beds were expected to be mature and so would better reflect the likely hydraulic limits in practice. In order to see if these can be elevated, the study assessed the impact of resting the systems whilst applying a fixed hydraulic rate during feeding periods. Direct analysis of the clogging potential was assessed through monthly monitoring of hydraulic drainage times combined with performance profiling of VFWs operating under varying conditions, providing a mechanical insight into the influence of applying resting periods to alleviate clogging.

Chapter 7: Tertiary vertical flow constructed wetlands: Understanding the impact of design choices on the potential economic viability in meeting tight ammonia discharge standards on small works

The optimal operational outcomes determined by the pilot plant trials (chapter 4, 5 and 6) for the proposed VFW design were utilised as the basis of an economic analysis to ascertain the economic feasibility of using VFW for tertiary treatment. The cost model developed for the VFW was benchmarked against alternative options to ascertain the potential case for adoption of the technology in the future and the key areas for further development to enhance the economic attractiveness of the technology.

Table 1-2 Thesis structure outlining the objectives addressed.

Chapter		Title	Objective met
1	Introduction		
2	Literature Review	Vertical flow constructed wetlands for municipal wastewater treatment: A review to assess the potential for use as a tertiary treatment process.	1
3	Case Study	Low energy tertiary treatment with vertical flow wetlands: a UK case study	1
4	Pilot plant- HLR	Impact of loading rate on the operation and performance efficiency of pilot scale tertiary vertical flow wetlands	2
5	Pilot plant- DF	Impact of dosing frequency on the nitrification potential within tertiary vertical flow wetlands	3
6	Pilot plant- RP	Impact of resting periods on the hydraulic behaviour and performance potential within tertiary vertical flow wetlands	4
7	Business case	Tertiary vertical flow constructed wetlands: Understanding the impact of design choices on the potential economic viability in meeting tight ammonia discharge standards on small works	5
8	Discussion		
9	Conclusions		
10	Future Work		

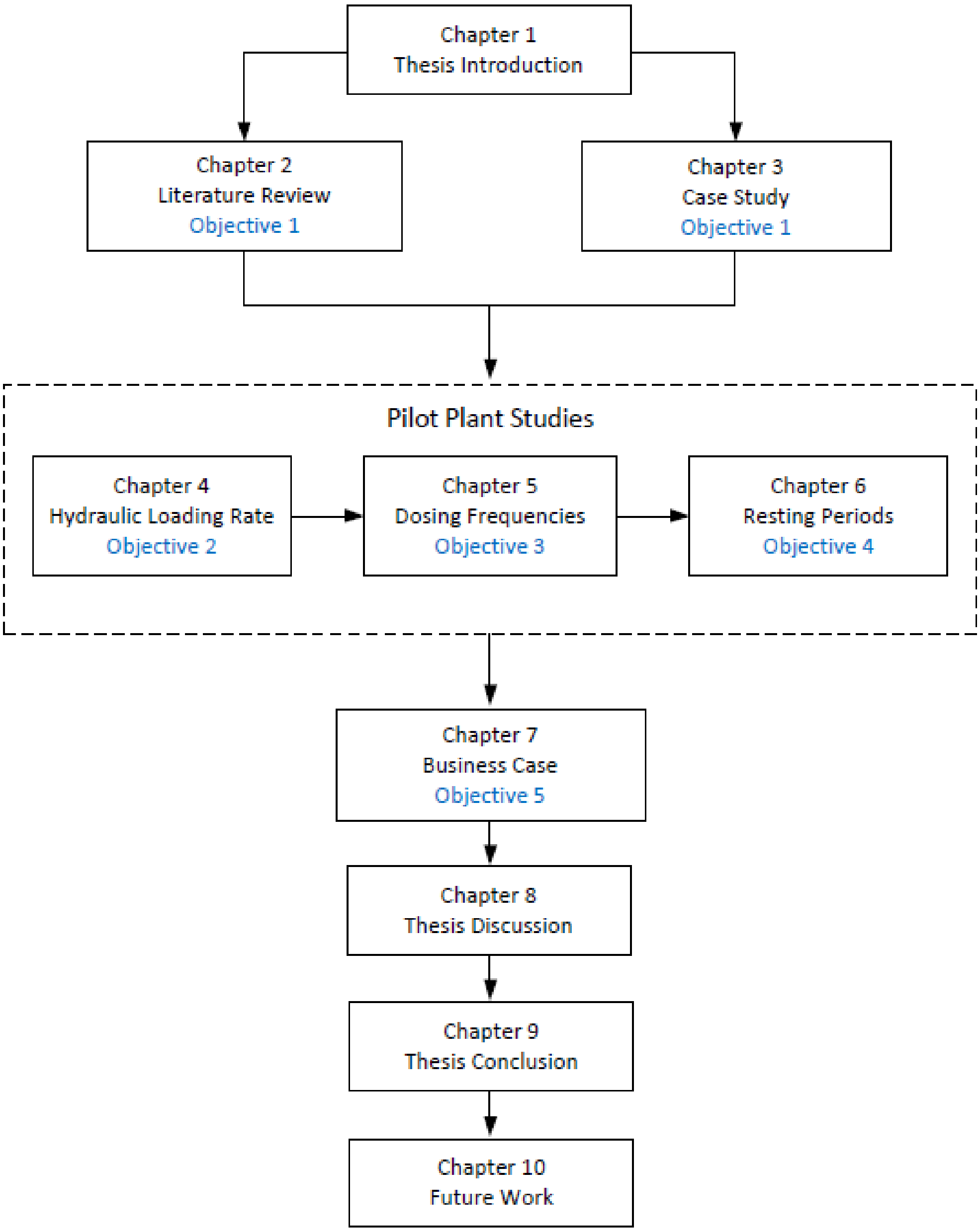

Figure 1-2 Thesis structure, interactions between the thesis chapters and the objectives met in each chapter.

1.4 References

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Chapter 2

Vertical flow constructed wetlands for municipal wastewater treatment: A review to assess the potential for use as a tertiary treatment process.

2. Vertical flow constructed wetlands for municipal wastewater treatment: A review to assess the potential for use as a tertiary treatment process.

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Abstract

Vertical flow wetlands are naturally aerated wastewater treatment systems that could provide an ideal solution to achieving challenging discharge consents, in the form of effluent polishing. Limited information is currently available within the literature regarding vertical flow wetland potential within tertiary application. Therefore this review considers the design, operation and treatment potential of vertical flow wetlands applied to whole and secondary treatment of municipal wastewaters with the aim of identifying key removal mechanisms and their associated influential design factors and operational strategies that could be adapted for vertical flow wetlands under tertiary application. The review concludes with an outlook for the potential of tertiary treatment vertical flow wetlands, outlining key design and operational considerations for future implementation.

2.1 Introduction

Constructed wetlands, or reed beds, are engineered passive (or near passive) planted media beds through which wastewater flows. The simplest and most common way to classify wetland systems is related to the location of the water table and the direction of flow. Accordingly, the main divider is water level with systems operating as either surface flow, where the water is visible above the media, or subsurface flow. In the case of the latter, the flow direction can be vertical (VF) or horizontal (HF). In general, HF systems tend to be operated continuously under fully saturated conditions such that the predominating conditions within the bed are anoxic or anaerobic enabling a degree of denitrification and solids degradation, and are widely used for secondary and tertiary applications. Recent adaptations have seen forced aeration or tidal operation to alter the redox conditions to enable nitrification (Butterworth et al., 2013). In contrast, most VF systems are operated with periodic batches of feed throughout the day whereby the bed is flooded and then allowed to drain (unsaturated down flow). Consequently, the bed is predominately aerobic and hence provides both organic and ammonia removal. During the feed and drain portions of the cycle, contaminants within the feed are captured and/or adsorbed into the biofilm surrounding the media for treatment during the unsaturated portion of the cycle (Paing et al., 2015).

The dosing of wastewater onto the bed results in the accumulation of retained solids and the generation of new biomass through microbial uptake of the available organics. Accordingly, the bed progressively accumulates solid loads which, if not managed, results in an increase in hydraulic resistance and potential subsequent clogging. In part, the accumulation of solids is managed by the aptitude of the active biofilms to mineralise the trapped material such that the clogging potential is ultimately a balance between the loading rate of new solids and organics, and the ability of the biofilm within the bed to process the load. Critically, the aerobic conditions must be maintained sufficiently to drive adequate mineralisation of the accumulating solids to stabile the system. This is achieved through management of the hydraulic loading rate, dosing frequency and the inclusion of rest cycles where the beds are left for several days to process the accumulated material (Molle et al., 2006; Torrens et al., 2009).

The ability to generate aerobic conditions passively has seen vertical flow wetlands (VFW) successfully used for raw and secondary wastewater treatment (Table 2.1) as well as for treatment of industrial wastewaters (Haberl, Perfler and Mayer, 1995), domestic wastewaters (Brix and Arias, 2005a), agricultural wastewaters (Kantawanichkul et al., 2003), landfill leachate (Spraggs et al., 2009) and for sludge treatment and drying (Uggetti et al., 2010). The design and operation of VFW are adapted to meet the challenges associated with the specific feed water characteristics as well as local conditions and the desired treatment goals. The main variables in relation to VFW design and operation are the media size used, the hydraulic loading rate, the dosing frequency and the inclusion of prolonged resting cycles.

Table 2-1 Examples of the applications of Vertical Flow Wetlands in wastewater treatment.

Application	system type	Purpose of treatment	Typical performances	Typical Loading		References
Whole Treatment	First Treatment Stage (rotational)	Solids and organic	COD: 82-88% TSS: 89-94% TKN: 60-69%	1.2m^2/pe	0.37m/d	Molle *et al.,*2005; Paign & Voisin, 2005
	Second Treatment Stage (rotational)	Solids, organic and nitrification	COD: 52-60% TSS: 59072% TKN: 78%	0.8m^2/pe		Molle *et al.,*2005; Paign & Voisin, 2005
Secondary Treatment	compact	solids, organic and nitrification	COD: 94% TSS: 80-92% NH_4-N: 78-91%	3m^2/pe	0.05m/d	Brix & Arias, 2005; Weedon, 2010
	hybrid (VF & HF)	nitrification and denitrification	COD:82- 86% TSS: 51% TN: - 6-24%			Foladori *et al.,* 2012; Ghrabi et al., 2011
Tertiary Treatment	Rotational	nitrification and polishing	COD: 55% TSS: 67% NH_4-N: 78-87%	0.22m^2/pe	0.45m/d	Cooper *et al.,* 1997
	compact			0.75-3.0m^2/pe	0.07-0.27m/d	Schönerklee *et al.,* 1997

2.2 Media

Wetland media plays a key role in the major removal mechanisms within a VFW, such as filtration and sedimentation, microbial uptake and interactions, and adsorption and precipitation. Therefore care and consideration must be taken in selecting the correct type and size of media for the desired VFW application and treatment type, in addition to VFW depth, treatment area and distribution of media down through the bed (Knowles et al., 2011; Stefanakis, Akratos and Tsihrintzis, 2014). Typically, VFW are designed to a depth of around 1m, with a treatment area of between 0.8-4m^2/pe and use three main media layers. Primary treatment VFWs, have an uppermost, main treatment layer of fine gravel (diameter: 2-8mm), with a depth of between 0.3-0.6m; an intermediate/ transition layer of pea gravel (diameter: 5-20mm) with a depth of between 0.1-0.4m; and a lower layer for drainage consisting of coarse gravel or cobbles (diameter: 15-50mm) with a depth of between 0.1-0.2m. Secondary and tertiary VFWs utilise sand (diameter: 0-4mm) with a depth of between 0.25–1.0m for the uppermost, main treatment media layer, however the transition and drainage media and depths remain the same as for the primary VFWs. (Brix and Arias, 2005a; Paing and Voisin, 2005; Prost-Boucle and Molle, 2012).

The smaller the media size the better the efficacy of filtration, and as such, sand based VFWs are expected to perform better than gravel based systems. To illustrate, a recent study comparing the two revealed removal efficiencies of: TSS >85% and >0% , COD >91% and >47% and BOD >96% and >39% for sand and gravel based VFWs, respectively (Bohórquez et al., 2017) (Table 2-2). The effectiveness of gravel based systems can be enhanced through lowering the hydraulic loading rate to ≤0.02m/d (Abou-Elela and Hellal, 2012). While sand provides a better treatment performance it also has a greater propensity to clog, such that some researchers have considered inverted systems with gravel above the sand layer (Song et al., 2015; Zhao et al. 2004). This has enabled an increased solids loading rate to be applied from 150gTSS/m^2/d in a typical VFW design to 250gTSS/m^2/d in an inverted design, whilst both maintaining the same removal efficiency in a pilot scale tidal flow VFW (Zhao et al. 2004). Additionally, during the same study, a reduced bed clogging time of 17 days was observed for the inverted VFW, compared to a clogging time of 6 days in the typical VFW design. This was believed to be due to the increased effective solids storage capacity and increased air convection and diffusion within the large pore spaces of the gravel media, promoting decomposition and mineralisation of the trapped solids. However, removal efficiency of TSS, COD, BOD_5, NH_4-N and TP remained comparable between the typical design and inverted VFW systems. Conversely, Song *et al.* (2015) reported a general improvement in the performance of the inverted VFWs compared to typically designed VFWs, in terms of nitrification and COD degradation due to improved dissolved oxygen (DO) concentrations through the depth of the bed. When considering performance in individual media layers, NH_4-N oxidation was greatest in the sand layer of the typical designed VFW. However, Song *et al*. (2015) concluded that typically designed VFWs are more advantageous over inverted systems, due to their reduced biofilm accumulation and strong clogging mitigation potential after bed resting.

The media also plays an important role with respect to biofilm development and activity as it provides the environment for attachment and growth, and impacts on the distribution of oxygen within the bed. The main removal mechanism for elimination of nitrogen forms from wastewaters is through microbial uptake and transformations. The substratum used within the VFW provides the surface to which biofilms can attach, with previous research showing successful pollutant removal is better performed by micro-organisms that are existing within a biofilm community (Huang et al., 2013). A biofilm is initially established when a single bacteria adheres to the surface of a substratum using Van der Waals forces, but quickly anchors itself by secreting a self produced extracellular polymeric substance (EPS), a sticky matrix composed of polysaccharides, proteins and extracellular DNA (Donlan, 2002). The secreted EPS allows and encourages the attachment of other bacteria, promoting the growth and development of the biofilm over the surface of the substratum. The adhesion of the biofilm and it's successful colonisation is thought to increase with increased surface roughness of the substratum (Donlan, 2002). Biofilm development is most effective within the main treatment layer, as this typically has the smallest media, providing the largest surface area for biofilm attachment and growth. However, care must be taken to ensure media is large enough to prevent clogging caused by biofilm blocking the available pores between media which, in time, could cause hydraulic overloading and reduced oxygen transfer through the bed (Song et al., 2015). Nitrification is a naturally occurring reaction in VFWs, whereby NH_4-N is oxidised into NO_2-N and then into NO_3-N by nitrifying bacteria within biofilms, in a two-step process as follows:

Step 1:

$$NH_4^+ + 1.5O_2 \rightarrow 2H^+ + NO_2^- + H_2O \tag{2-1}$$

(Ammonium-nitrogen to Nitrite-nitrogen)

Step 2:

$$NO_2^- + 0.5O_2 \rightarrow NO_3^- \tag{2-2}$$

(Nitrite-nitrogen to Nitrate-nitrogen)

For successful nitrification, the nitrifying bacteria require a sufficient supply of nutrients in the feeding wastewaters and an optimum dissolved oxygen concentration of 3-4mgO_2/L (Song et al., 2015). For nitrate-nitrogen removal from wastewaters, transformation into nitrogen gas must be achieved through a step-wise nitrate reduction reaction (Equation 2-3), for which a denitrifying biofilm is required. Denitrification is an anaerobic process, and as VFWs predominantly operate under aerobic conditions their potential to denitrify is limited.

Denitrification shown through a nitrate reduction reaction:

$$NO_3^- \rightarrow NO_2^- \rightarrow NO \rightarrow N_2O \rightarrow N_2 \tag{2-3}$$

(Nitrate to Nitrite, to Nitric Oxide, to Nitrous Oxide, to Nitrogen Gas)

Comparison between sand and gravel show respective NH_4-N removal efficiencies of 77% and 36% with predominant concentrations of nitrite in the gravel system and nitrate in the sand based system (Bohórquez et al., 2017). In typical VFW designs, DO decreases with depth, often to below 0.5mgO_2/L, indicating a potential inhibition of nitrification. In some cases, media with specific sorption capacity for ammonia have been used, such as zeolites, and these generate enhanced biofilm attachment and development (Stefanakis and Tsihrintzis, 2012). For instance, comparison between volcanic rock and natural zeolite showed a respective NH_4-N removal efficiency of 69.6% compared to 91.6% (Huang et al., 2013). Analysis of the biofilm on each of the two media identified a more diverse biofilm community on the zeolite media compared to the volcanic rock with the dominating ammonia oxidising bacteria (AOB) switching from nitrosospira in the zeolite to nitrosomonas within the volcanic rock. Similar findings have been reported elsewhere (Bruch et al. 2011; Canga et al. 2011) demonstrating the potential for use of zeolites in tertiary applications where variable and potentially low loads are expected (Butterworth et al., 2016).

The use of specialised media is more prevalent in the case of phosphorous (P) and heavy metals (HM), as the main removal pathways are thought to be adsorption and precipitation (Vymazal, 2007). Gravel based VFWs have been shown to have a low phosphorus treatment capacity with reported removals of 4% (Korkusuz et al., 2005), whilst sand based VFWs have achieved phosphorus removals of between 39-64% (Prochaska et al., 2007). However, Luderitz and Gerlach, (2002) reported a decrease in P removal efficiency from 44% to 27% in 6 years of operation when using a mixed media of sand and clay. Instead media rich in aluminium, iron, calcium or magnesium oxides are preferred as these drive a reactive step to precipitate dissolved species (Arias et al., 2001; Seo et al., 2005). The main limiting factor of using media for phosphorus removal is that once the P-binding sites on the media become saturated, P removal diminishes and the media has to be replaced or regenerated (Drizo et al., 2002). Reported media that enhance phosphorus removal in VFWs include steel and blast furnace slag, zeolite, bauxite, dolomite, shale and apatite as well as more innovative materials such as oyster shell (Seo et al., 2005) and broken bricks (Wang et al., 2013). The most important parameter for P removal with media is the calcium and calcium oxide content within the substrate, which enhances precipitation and shows a positive correlation with P retention (Arias et al., 2001; Wang et al., 2009). However, when used at full scale, it has been predicted that calcium rich media will become P saturated within 12 months of operation (Arias et al., 2001), and it has therefore been recommended to mix the calcium rich media, such as calcite and apatite, with the usual sand or gravel substrate (Brix et al., 2001; Molle et al., 2005b). As VFW operate intermittently with free drainage, it has been hypothesized that due to the low residence time of wastewater in VFWs, phosphorus has limited contact time with the surface of the media. Therefore low levels of P adsorption onto the media has resulted in low P removal, particularly when compared to wetlands normally operated under saturated conditions (Brooks et al., 2000; Drizo et al., 2002; Stefanakis and Tsihrintzis, 2012).

Table 2-2 Removal efficiencies (%) of VFW under different applications

Chapter section	Reference	Design	TSS	COD	BOD_5	NH_4-N	TP
2.2 Media	Bohorquez et al., 2017	Sand	85-89	91-93	96-97	77-83	-
		Gravel	0	47-59	39-59	36-45	-
	Zhao et al., 2004	Typical design	-	71	66	34	51
		Inverted design	-	74	67	33	51
	Song et al., 2015	Typical design	-	46-55	-	28-49	-
		Inverted design	-	49-60	-	39-56	-
	Huang et al., 2013	Volcanic Rock	-	46	-	70	70
		Zeolite	-	51	-	92	81
	Korkusuz et al., 2005	Gravel	59	44	-	53	4
		Blast Furnace Slag	63	47	-	88	45
	Luderitz and Gerlach, 2002	Sand and clay	-	-	-	-	27-44
2.5.1 Whole Treatment	Lana et al., 2013	Whole treatment- planted	83-84	72-80	79-82	56-60	35-45
		Whole treatment - unplanted	77-78	72-82	78-83	49-55	30-50
	Molle et al., 2005	Whole treatment	95	90	-	85	-
	Paing and Voisin, 2005	Whole treatment	98	94	99	-	45
	Prigent et al., 2013	Whole treatment	93-95	88-93	95	-	-
2.5.2 Secondary Treatment	Abou-Elena and Hellal, 2013	Secondary treatment	92	88	90	-	=
	Brix and Arias, 2005	Secondary treatment	91	-	92	78	25
	Gikas and Tsihrintzis, 2012	Secondary treatment	-	94	96	93	79
	Haberl et al., 1995	Secondary treatment	-	90	96	94	63
	Mietto and Borin, 2013	Secondary treatment	-	92	-	94	27
2.5.3 tertiary Treatment	Cooper et al., 1997	Tertiary treatment	63-67	-	64-82	52-80	-
	Schönerklee et al., 1997	Tertiary treatment	-	49-62	-	43-91	3-65
	Toscana et al., 2009	Tertiary treatment	-	63	-	43	-

2.3 Vegetation

Nutrients, such as nitrogen and phosphorus, are typically found in most wastewaters and are an essential requirement for the growth and development of all plant species. These nutrients along with heavy metals such as lead, zinc, cadmium, chromium, nickel, copper, iron, mercury and arsenic, are taken up in the wastewater through the plants root system and distributed to other parts of the plant in small amounts (Stefanakis et al., 2014). To fully utilise plants as a direct mechanism for nutrient and heavy metal removal from wastewaters, it is recommended to use plants with rapid growth, as taller plants have a greater tissue storage capacity (Brix, 1994; Vymazal et al., 1998). It is common practice in Europe to use *Phragmites australis* (the common reed) in wetlands for wastewater treatment purposes, as they can grow up to 4m in height during a growing season (between April and June) (Brix and Arias, 2005a; Cooper and Green, 1995). Additional studies have shown other emergent plants, including Juncaceae (rushes), Scirpus *spp*. (bulrushes), Glyceria *spp.* (mannagrasses) and particularly Typhaceae (Cattail) to also be beneficial in wastewater treatment as they possess an extensive root system and have an affinity to assimilate nutrients from the water, and additionally can be used as an alternative wetland plant in areas where invasive Phragmites are deemed a threat to native species. For sufficient plant coverage over the wetland surface, a planting density of 4 plants per m^2 is recommended (Brix and Arias, 2005a; Cooper and Green, 1995). Pollutant uptake by the plant is most effective during the growing season, when the plants needs are at their greatest. To reduce the level of pollutants leaching back into the system through decomposition and mineralisation of dead plant material, annual harvesting of the above ground matter is recommended at the end of each growing season (Vymazal et al., 1998). However the majority of nutrients and heavy metals up-taken by the plants are retained within the below ground biomass, therefore some leaching is to be expected (Lesage et al., 2007; Zhang, Rengel and Meney, 2007). Harvesting of the above ground matter is achieved through cutting back or trimming of wetland plants close to the bed surface, with consideration to avoid damaging above ground pipework.

Harvesting of wetland plants is generally only considered viable for small-scale systems due to the associated costs of plant material disposal and lack of opportunity for product recovery (Kadlec and Wallace, 2009).

In addition to the direct uptake, plants also provide additional surface area for microbial and biofilm establishment through their extended root system, enhancing the potential of pollutant removal through microbial to wastewater interactions (Vymazal et al., 1998). This is particularly beneficial to nitrifying organisms as oxygen, produced during plant photosynthesis, is transferred from the leaves of the plant to the root system then into the rhizosphere, making it an oxygen rich environment during daylight hours and providing optimal conditions for nitrification. The deep and complex root system of the emergent plants can also enhance contact time between the wastewater and the contaminant removal mechanism components (media, microbial communities and plant root system), potentially increasing nutrient removal, by contributing towards water velocity deceleration through the wetland, prevention of clogging within the wetland media and preservation of the systems hydraulic conductivity (Brix, 1994). On wetlands used for primary wastewater treatment, the plants can provide 'wind-rock' - a circular indentation- within the surface sludge layer, which increases water permeability and infiltration rate, and preserves the hydraulic conductivity as the water flows through the wetland following a path provided by the root system. In a comparative study on these systems, the absence of emergent plants created an excess of surface water due to the decreased hydraulic conductivity and poor infiltration rates of the sludge deposit layer (Molle et al., 2006).

Emergent plants are known to be beneficial to the overall wetland system, with studies proving planted wetland systems have a more effective pollutant removal than unplanted systems (>30% greater nitrogen removal in VFW planted with Phragmites compared to unplanted systems and >5% greater phosphorus removal), and minimal differences between the different plant species (Abou-Elela and Hellal, 2012; Macci et al., 2014; Zhang, Rengel and Meney, 2007). However, despite having the ability to take up the aforementioned pollutants, the effectiveness of direct nutrient uptake by emergent plants is negligible compared to alternative removal mechanisms, and is therefore not considered a primary mechanism of removal (Stefanakis and Tsihrintzis, 2012).

2.4 Loading Rate

The hydraulic loading rate (HLR) has a strong impact on the performance of the bed through direct hydraulic impacts as well as the associated link to organic and solids loading. Increasing the HLR consequently increases the batch volume dosed per feed, resulting in an elevated water velocity, and therefore reducing the hydraulic retention time and potential contact time between the pollutants in the wastewater and the media/biofilm/plant roots (Torrens et al., 2009). However, the increased velocity also results in thinner hydrodynamic boundary layers surrounding the biofilm (Donlan, 2002), providing quicker and easier diffusion of oxygen into the biofilm. Alternatively, if the greater HLRs are applied through increasing the number of feed cycles or by increasing the feed lengths, oxygen transfer may decrease due to lack of VFW recovery time (Molle et al., 2006). The converse is true for lower HLRs but there is potential for the creation of preferential pathways through the media which can reduce the treatment capacity of the bed if the HLR is too low to allow complete surface coverage of wastewater during a feed (Cooper, 2005).

Hydraulic loading rates vary between VFW applications. First stage of whole treatment VFW, typically comprising three separate beds and operated rotationally, receive an average HLR of around 0.12m/d, equating to a HLR of 0.37m/d for the operating VFW (Molle et al., 2005a; Paing and Voisin, 2005; Prigent et al., 2013a). Second stage of whole treatment and secondary treatment VFW receive a HLR of around 0.20m/d (Brix and Arias, 2005b; Langergraber et al., 2007; Mietto and Borin, 2013; Molle et al., 2005a; Song et al., 2006), but can be as low as 0.05m/d (Gikas and Tsihrintzis, 2012). Hydraulic loading rates for tertiary treatment VFWs have been reported as 0.27m/d for parallel systems, whereby four beds designed in parallel were operated simultaneously (Schönerklee et al., 1997) and 0.45m/d across four rotational beds (Cooper et al., 1997). The HLRs need to be adjusted to local climatic conditions with the recommendation that a reduced HLR of between 0.05-0.10m/d, is applied in colder climates (Stefanakis et al. 2014).

Another influential factor when considering HLRs of VFWs is its impact on the solids and organic loading rates (OLR), as these can lead to adverse effects involving clogging (Winter and Goetz, 2003). Stefanakis *et al.* (2014) recommend an OLR of up to 30gCOD/m^2/d for colder climates and up to 80gCOD/m^2/d in warmer climates. A clogging study conducted by Winter and Goetz (2003) conclude OLRs should not exceed 20gCOD/m^2/d and solids loading should not exceed 5gTSS/m^2/d, for single stage secondary treatment systems operating in Central Europe climatic conditions without rotation. However Torrens *et al.* (2009) did not report any clogging issues on a secondary treatment VFW in France operating under a HLR of 0.8m/d and an OLR of 170gCOD/m^2/d.

Research conducted to show how the HLR effects the treatment performance of first stage whole treatment VFW systems have shown successful removal of COD and TKN up to an HLR of 0.6m/d (Paing et al., 2015). Hydraulic loading rates above this showed variability and a decrease in performance efficiency. Removal of TSS and BOD remained consistent for HLRs of up to 1.0m/d, however this is believed to be due to a dilution effect. Molle *et al.* (2006) reported a decrease in nitrification potential in VFWs receiving hydraulic loads of over 0.8m/d, believed to be due to insufficient oxygen transfer into the VFW in between feeds.

2.5 Dosing and Resting Periods

Similarly to HLR, the daily dosing frequency (DDF) can be manipulated to control treatment performance and reduce filter clogging risks (Molle et al., 2006). When applying a single specified HLR, reducing the DDF is likely to increase the batch volume and infiltration rates, increase the oxygenation, but reduce the hydraulic retention time and wastewater to biofilm/media/plant root contact time. Additionally a lower DDF provides longer rests between feeds, allowing the media to dry and therefore reducing the effective reactor volume (Molle et al., 2006; Torrens et al., 2009). Vertical flow wetlands receiving a low DDF may have a reduced treatment capacity as contact time is effectively poorer with larger batch volumes. For instance, Molle *et al.* (2006) reported poor COD removal in VFW receiving a low DDF, despite a sufficient oxygen supply for mineralisation, but good nitrification due to adsorption of NH_4-N onto trapped OM followed by nitrification during the rest stage of the intermittent feed. However, this method for nitrification is unsustainable and efficiency decreases with time. Torrens *et al.* (2009) also reported low COD removal in VFW with low DDF, with poor removal rates also observed for TSS and KN. Additionally it was determined that nitrification is optimised during the recovery periods between feeds, which is in agreement with previous findings. Additional research on dosing frequencies in VFW indicated that systems receiving a low DDF are likely to develop an even biofilm growth over the whole VFW surface and depth, which will enhance the longevity of the system (Bancole et al., 2003).

In contrast, when applying a single specified HLR, increasing the DDF will reduce infiltration rates and therefore increase wastewater retention and contact time between pollutants and media/biofilm/plant roots. With increased DDF, the water content retained within the media is greater therefore increasing the effective volume of reaction (Molle et al., 2006). Oxygenation of systems operating under these conditions is limited due to a decrease of oxygen diffusion into the media during feed cycles and reduced rest periods between feeds (Torrens et al., 2009). Treatment capacity for VFWs with a high DDF includes good COD, TSS and KN removal (Torrens et al., 2009) in addition to good organic matter and nitrogen removal (Bancolé et al., 2003). However nitrification potential was low, due to the reduced oxygen levels (Molle et al., 2006; Torrens et al., 2009). Additionally, Bancole *et al.* (2003) found that in systems operating with a high DDF, biofilms tend to accumulate within the top 10cm of the VFW surface media, potentially having a negative impact on the hydraulic conductivity of the water through the system, the infiltration velocities and the oxygen diffusion potential. Torrens *et al.* (2006) reported that operation of systems receiving high DDFs should be limited to 3-4 consecutive days use, before applying a resting period of 7 days to allow the system to recover and to restore oxygen transfer capacity and nitrification potential.

Resting periods are commonly applied to systems receiving high solids and organic loadings with the VFWs operated on a rotational basis, thus limiting system disruption, as demonstrated by the French two stage systems (Molle et al., 2005a). Here the operating bed is rested for around double the length of time of the feeding period. This provides time for complete drainage of accumulated surface water and mineralisation of the deposited surface sludge blanket and trapped organic matter within the media pores, which in turn reduces amount of clogging matter and alleviates clogging potential, whilst ensuring continued oxygen transfer to within the VFW (Chazarenc and Merlin, 2005; Molle, 2014). Resting periods can also be incorporated into regimes of continually operated VFWs to remediate clogging or prevent future clogging events (Knowles et al., 2011).

2.6 Applications for Wastewater Treatment

2.6.1 Whole Treatment

Vertical flow wetlands for the treatment of raw wastewater in small rural communities has been an area of interest in France since the late 1970's, when the Centre d’etude du Machinisme Agricole du Génie Rural des Eaux et Forêts (CEMAGREF) (now known as the National Research Institute of Science and Technology for Environment and Agriculture (IRSTEA)) constructed the first wastewater treatment wetlands in France (Boutin, 1987). These original systems were designed in accordance with Dr Seidel's patented process and comprised a five step multi-level treatment including two stages of vertical flow wetlands ('percolation flow'), operated in parallel, followed by three stages of horizontal flow wetlands ('translational flow') operated in series (Lienard and Cedex, 1988). Further research by CEMAGREF saw the optimisation of both design and operation of the treatment system, which was characterised in their 1998 publication of recommendations, whereby it was suggested that only two stages of rotational vertical flow wetlands were required for sufficient complete wastewater treatment (Boutin, 1998). By rotating the operation between the VFWs, a prolonged resting period can be applied, aiding in VFW recovery. To date, France has been the leading country in utilising vertical flow wetlands as a viable option for the whole treatment of wastewater in small rural communities, with more than 2500 systems currently operating in France, with the majority designed and operating according to CEMAGREF's guidelines. However, more recently, countries including Brazil and the UK are installing systems to mirror CEMAGREF's first stage and whole system recommendations, respectively (ARM, 2014; Cota et al., 2011; Lana et al., 2013). Typically, these designs require three or four first stage wetlands operated on a rotational basis, with one VFW receiving influent whilst the other beds are resting. The effluent from the first stage VFWs is then siphoned or pumped onto one of two or three second stage VFW, which are also operated rotationally. The recommended total active treatment area for these systems is $2m^2$/pe, of which $1.2m^2$/pe is required for the first stage and $0.8m^2$/pe for the second stage, with a typical hydraulic loading rate of $0.37m^3/m^2/d$ on the active wetland, provided over 10-15 intermittent daily feeds (Molle et al., 2005; Paing and Voisin, 2005) (Table 2-3). The first stage VFW contains a coarser media than conventional VFWs and second stage, usually fine gravel (ø2-8mm) opposed to sand, which reduces potential for biological clogging and controls biomass growth. As the treatment systems receive screened raw sewage, a deposited sludge layer ultimately builds up on the wetland surface, but is controlled by applying a feed/ rest regime, commonly in a 1:2 feed/rest ratio (ie: one week feed: 2 week resting period). This resting period provides a sufficient recovery time for the wetlands, allowing degradation and mineralisation of the surface sludge layer and of captured solids within the media layers, resulting in a surface sludge accumulation rate of 1-2cm per annum requiring removal once every 10-15 years (Chazarenc and Merlin, 2005; Troesch et al., 2014). First stage wetlands are planted with Phragmites which enhances sludge mineralisation by increasing aeration within the sludge through stem and root movements (Nielsen, 2003; Uggetti et al., 2010). Second stage wetlands are usually planted with Phragmites or other reed species, however some studies have indicated that plants do not play a significant role in treatment performance on second stage wetlands (Paing and Voisin, 2005).

The formation of the deposit sludge layer occurs due to the progressive accumulation of suspended solids and particulate organic matter in influent wastewaters in addition to reed detritus and microbial biomass. The deposit layer acts as an additional filtration matrix, improving solid and OM removal from influent wastewater due to the much smaller pore sizes within the sludge deposit, but also has a positive effect on the hydrology and biological activity of the VFW by enhancing water retention, improving water distribution and percolation times, and providing a specific microbiological growth support system (Chazarenc and Merlin, 2005; Molle, 2014). However, to prevent surface clogging occurring, which can potentially limit oxygen transfer, the operation of the VFW, such as intermittent loading regime and recovery periods between feeds, must be optimised to control hydraulic and organic loading to favour mineralisation of the deposit layer (Molle, 2014). Typical performances of whole treatment systems are between 87-99% for suspended solids, 80-94% for COD removal, 84-99% for BOD removal and 52-82% for ammonia (Lana et al., 2013; Molle et al., 2005; Paing and Voisin, 2005; Prigent et al., 2013b) (Table 2-2). In a recent survey of operating two stage systems for coarsely screened raw sewage, the average effluent qualities were 10±10mgTSS/L, 6±4mgBOD_5/L and 5±6mgNH_4-N/L for TSS, BOD_5 and NH_4-N respectively, based on composite samples (Paing et al, 2015).

One of the main disadvantages of using whole treatment VFW systems are that they require a relatively large land footprint (2m^2/pe) when compared to other (more energy intensive) wastewater treatments such as activated sludge process and oxidation ditches (0.2m^2/pe) (Koot and Zeper, 1972), thereby increasing the capital cost (Cota et al. 2011; Lana et al. 2013; Prigent, et al., 2013a). Another drawback is the limited potential to remove total nitrogen (Prigent et al. 2013b). Novel adaptations to resolve this issue include incorporation of a 'double tiered' treatment area and including a saturated layer (Silveira et al., 2015) and the inclusion of recirculation (Prigent et al., 2013b; Prost-Boucle and Molle, 2012).

Table 2-3 Design factors of Vertical Flow Wetlands for whole, secondary and tertiary treatment

Reference	Application	System Type	Area	HLR	DDF	Vegetation	Depth	Type	Diameter
Paing & Voisin, 2005.	First Stage	rotational	1.2m²/pe	0.12m/d	10-15/day	*Phragmites australis*	0.50m	gravel	2-8mm
							0.20m	gravel	10-20mm
							0.20m	gravel	20-40mm
	Second Stage	rotational	0.7m²/pe	0.2m/d	10-15/day	*Phragmites australis*	0.40m	sand	0-4mm
							0.30m	gravel	3-8mm
							0.20m	gravel	10-20mm
Lauschmann et al., 2013	First Stage	compact	4m²/pe	0.01m/d	-	*Phragmites australis*	0.50m	sand	0.06-4mm
							0.10m	gravel	4-8mm
							0.20m	gravel	16-32mm
	Second Stage	compact	1m²/pe	0.06m/d	-	*Phragmites australis*	0.50m	sand	0.06-4mm
							0.10m	gravel	4-8mm
							0.20m	gravel	16-32mm
Giakas & Tsihrintzis, 2012	Secondary	compact	3m²/pe	0.05m/d	8/day	*Phragmites australis*	0.70m	gravel	2-10mm
							0.30m	gravel	20-40mm
Mietto & Borin, 2012	Secondary	compact	0.7m²/pe	0.2m/d	96/day	*Phragmites australis*	1.90m	expanded clay	8-16mm

Reference	**Application**	**System Type**	**Area**	**HLR**	**DDF**	**Vegetation**	**Depth**	**Type**	**Diameter**
Torrens et al., 2009	Secondary	rotational	1m^2/pe	0.2-0.8m/d	4-32/day	*Phragmites australis*	0.65m	sand	-
Weedon, 2010	Secondary	compact	2m^2/pe	0.05m/d	-	Reed	0.80m	sand	-
							0.05m	gravel	10mm
							0.15m	gravel	40mm
Ghrabi et al., 2011	Secondary	hybrid (HF-VF-HF)	2.4m^2/pe	0.02m/d		*Phragmites australis*	0.50m	sand	0.2-1mm
							0.10m	gravel	5-10mm
							0.15m	gravel	40-70mm
Tuszynska & Obarska-Pempkowiak, 2008	Secondary	hybrid (HF-VF-HF)	2m^2/pe	0.03m/d	-	*Phragmites australis*	0.40m	sand	0.2-2mm
Cooper et al., 1997	Tertiary	rotational	0.6m^2/pe	0.3m/d	8/day	*Phragmites*	0.05m	sand	-
							0.35m	gravel	5-10mm
							0.30m	gravel	30-60mm
Schonerklee et al., 1997	Tertiary	compact	0.4m^2/pe	0.15m/d	4-12/day	*Phragmites australis*	1.20m	sand	0-4mm
							0.10m	gravel	4-8mm
							0.25m	gravel	16-32mm

2.6.2 Secondary Treatment

The use of VFWs for secondary application dominates all other VFW applications for wastewater treatment, and as such have more variation in design. Historically secondary VFWs took design inspiration from the earlier work of Seidel (1976), with several wetlands being built in parallel and operating on a rotational basis, typically in a multistage treatment process. The first example of such a system in the UK was at Oakland Park in 1987 (Burka and Lawrence, 1990). The wetlands at the site were installed to treat effluent from a septic tank and settlement chamber and comprised two VFW stages: four rotational VFWs in the first stage and three VFWs in the second stage, followed by a two stage horizontal flow wetland (HFW), totalling a treatment area of 1.4m^2/pe. These rotational systems are advantageous for reducing clogging caused by overloading (Griggs and Grant, 2000; Weedon, 2003), as the rotational operation enables sufficient time for mineralisation of the collected and entrapped solids, and the use of a larger, stone-type, main media material increases the hydraulic conductivity of the VFW. In addition, rotated beds can also receive higher loading rates (0.2m/d) than un-rested beds (0.045m/d) and are generally easier to construct due to their shallower total depth requirements (Brix and Arias, 2005a; Burka and Lawrence, 1990).

More recently, the focus of research has been on constructing individual VFWs that can be operated singularly or collectively in parallel or in series, depending on the type and level of treatment required, typically following sedimentation or septic tank treatment. Some of the earliest research on this type of system was carried out in Austria, whereby a five year study was conducted to optimise design and operation of both a one stage, singular VFW and two singular VFWs operated in series, using an intermittent feeding regime but without an extended 'recovery' period (Laber *et al.*, 1997; Perfler and Haberl, 1993). Since then this type of system using intermittent flows and deep sand media layers for main treatment has been adopted as the standard VFW design for secondary application, in many countries including Austria, Denmark and UK (Vymazal et al., 1998). As a trade-off for these single bed systems not needing resting periods, a larger area is required per person equivalent to prevent hydraulic over-loadings and inevitable clogging (Weedon, 2003). The earliest research on these systems suggested an area of 5m^2/pe and a very light hydraulic loading of 0.023m/d was needed to reach desired levels of treatment (Perfler and Haberl, 1993). However, more recent studies have shown VFW to achieve similar performances with an area of 3m^2/pe and hydraulic loading of 0.045m/d in Denmark (Brix and Arias, 2005a), and even with an area of 2m^2/pe and hydraulic loading of up to 1m/d in the UK (Weedon, 2003). Additionally, for these individual VFWs to achieve a greater performance efficiency, it is recommended that 50% of the wetland effluent be re-circulated to the primary treatment, and re-passed through the wetland, to enhance nitrate (and therefore total nitrogen) removal through denitrification and to further stabilise the treatment performance of the system (Brix and Arias, 2005a; Laber et al., 1997). Alternatively a shallow saturation level within the wetland media maybe introduced to promote anaerobic or anoxic conditions suitable for denitrification (Langergraber et al., 2009).

To optimise treatment efficiency, it is common to combine VFW treatment with other wetland types to form a hybrid wetland system. Configuration of these systems can be implemented to target the removal of specific pollutants, based on the desired outcome. For example, if nutrient removal was required, a VFW-HFW hybrid would provide aspects of nitrification, using the naturally aerated VFW to convert ammonia into nitrates, followed by denitrification using the predominantly anaerobic and anoxic HFW to convert nitrates into atmospheric nitrogen. For systems receiving heavier solids and organic loadings, a HFW-VFW hybrid could be used to remove suspended solids and COD/BOD and carry out partial denitrification in the first instance, followed by further removal of solids, organics and nitrification in the VFW (Cooper, 1999; Vymazal, 2005). However, if these systems require total nitrogen removal, recirculation of the treated effluent or the addition of supplementary HFW would be needed (Laber et al., 2003).

Typical removal efficiencies for secondary VFWs are between 43-92% for suspended solids, 48-94% for COD removal, 48-97% for BOD removal and 49-78% for ammonia (Abou-Elela and Hellal, 2012; Brix and Arias, 2005a; Foladori et al., 2012; Gikas and Tsihrintzis, 2012; Haberl et al, 1995; Mietto and Borin, 2013; Song et al., 2006; Tietz et al., 2008) (Table 2-2). Secondary VFW are typically constructed with fine media material, such as sand, meaning that pore spaces between the media are minimised and filtration is increased. Illustrative effluent quality for secondary VFWs are 10mg/L BOD and 5mg/L ammonia for a conventional VFW (Brix and Arias, 2005), and 20mg/L BOD, 30mg/L TSS and 20mg/L ammonia for a compact design (Weedon et al. 2016).

2.7 Outlook for Tertiary Vertical Flow Wetlands

Vertical flow wetlands for wastewater treatment have demonstrated a high treatment capacity in terms of solids and organic matter removal and has shown potential for successful removal of phosphorus and emergent pollutants such as heavy metals and pharmaceuticals, across all applications and under the umbrella of 'low-cost, low energy consumption and low maintenance'. More impressively, VFWs have a natural affinity for achieving high levels of ammonia removal through nitrification, particularly when operated under optimised conditions. This provides a positive outlook for utilising VFWs as a tertiary technology to polish secondary effluents prior to surface water discharges, in line with achieving the proposed tightened nutrients discharge consents defined by the Water Framework Directive (European Union, Water Framework Directive 2000/60/EC) (Griffin and Pamplin, 1998; Johnson, Camargo Valero and Mara, 2007).

Due to historical clogging issues resulting from poorly designed and operated tertiary VFW systems (Cooper et al. 1997), there is very limited literature available on the design, operation and treatment performance of VFW systems in tertiary application. A search within the existing literature identified only nine relevant papers on tertiary VFW, two of which are small experimental full scale systems (Cooper et al., 1997; Schönerklee et al., 1997), three are pilot plant scale systems, with one as a hybrid (Giraldi et al., 2009; Lahav et al., 2001; Toscano et al., 2009) and four are lab scale systems (Green, Friedler and Safrai, 1998; Reif et al., 2011; Wang et al., 2012; Zhang, Rengel and Meney, 2007). Of the two small full scale systems, one is a two stage rotational system with four VFW on each stage, which was installed in the UK in 1992 and was designed based on the work of Burka and Lawrence (1990). The other is a single stage, multi-bed system working as four parallel beds, installed in Austria in 1995 (Cooper et al., 1997; Schönerklee et al., 1997). The design of the two full scale systems varied greatly: Cooper *et al.* (1997) initially designed the two stage system based on a treatment area of $0.4m^2$/pe to receive a hydraulic loading of approximately 0.45m/d (averaged over a 7 day week). The wetland media mainly comprised a relatively shallow main treatment layer of gravel (35cm) topped with 5cm of sharp sand. Within two years of operation the effective number of person equivalents dropped from 290 to 200, increasing the treatment area to $0.62m^2$/pe. Schönerklee *et al.* (1997) investigated the effect of altering the hydraulic loading rate and therefore had an effective treatment area of between $1\text{-}3m^2$/pe with hydraulic loading rates ranging from 0.07-0.27m/d, However it was concluded that a treatment area of $1m^2$/pe was sufficient for tertiary treatment. The effect of main treatment media depth was also investigated; a shallow depth of 20cm and greater depth of 80cm were compared, both of which comprising an even mixture of sand (0-4mm) and gravel (4-8mm).

Comparing the VFW design within all nine papers shows a predominant use of sand and gravel as media treatment layers, with the exception of one paper using zeolite for main treatment media (Wang et al., 2012). The majority of these VFWs have a similar overall depth of between 60-75cm, with the exception of Lahav *et al.* (2001) using an exceptionally deep treatment media of 180cm, and Zhang *et al.* (2007) using a very shallow media at 15cm total depth. Most of the VFWs are planted with *Phragmites*, however three of the studies use multi-cultures of: *Canna indica* and *Schoenoplectus validus* (Zhang et al., 2007); *Cyperus papyrus, Canna sp., Iris pseudoacorus,* and *Juncus ensifolis* (Giraldi et al., 2009); and *Phragmites australis* and *Typha* (Wang et al., 2012). The few studies on tertiary application of VFWs have shown solids removal of 63% (Cooper et al., 1997), BOD removal of 78% (Cooper et al., 1997) and COD removal of between 18-62% (Schönerklee et al., 1997; Toscano et al., 2009).

Although VFWs are a widely accepted and commonly used wastewater treatment process, particularly within Europe, there are no definitive and generally accepted criteria for VFW design, construction and operation. Instead, VFWs are likely to be designed based on other successfully performing VFW systems in the same geographic location, that have been optimised to external factors such as: climatic conditions; land and resource availability; discharge points; applicable discharge consents and sensitivity of the receiving water course; treatment application; influent characteristics (Stefanakis, Akratos and Tsihrintzis, 2014). With that said, there are published guidelines available for whole and secondary treatment VFW design and operation for use in the UK, Denmark, Nepal, Austria and France (Table 2-4).

It is postulated that a VFW under tertiary application would receive a more diluted wastewater, with reduced solids, organics and nutrient concentration, compared to a secondary VFW receiving primary treated effluent (Table 2-5). Accordingly, it is anticipated that operation with an increased HLR is possible in order to reach the limiting loading conditions. Due to a lower nutrient availability in the influent to a tertiary VFW, biomass accumulation over the surface of the media would be reduced (thinner biofilms), consequently reducing the oxygen demand. This would provide the opportunity for VFW systems to be operated with a higher DDF and shorter rest time between feeding cycles. This would not only help in distributing the increased HLR evenly through the day, but could also potentially increase treatment efficiency of COD, TSS and N removal, as seen in secondary VFW (Torrens et al., 2009). In secondary VFWs, a low nitrification rate was reported in systems receiving a high DDF, although this was attributed to insufficient oxygen transfer and availability. However, it is postulated that VFWs in tertiary application would require a reduced oxygen demand for diffusion into the thinner biofilms, potentially increasing the capacity for nitrification. However, as greater DDFs typically correspond to small batch volumes and subsequent increased retention time, consideration must be given to allow sufficient drainage time between feeds to minimise likelihood of clogging events. Selection of the appropriate media size is balanced between maximising contact between the water and the biofilm and ability to operate at high HLRs. Ultimately this needs to be ascertained but the expectation is the smaller media will probably be beneficial to ensuring high level treatment down to low residual levels.

Table 2-4 Current VFW guidelines for construction

Reference	Sizing	Depth	Media	Distribution	Collection	Vegetation	Loading Rates	Treatment type
Griggs and Grant, 2000. 'Good Building Guide GBG 42 (part 1 and 2)' UK	4pe=2.0m^2/pe, 100pe=0.8m^2/pe	1-1.5m	1st layer (top): 0.10m washed sand (0.2-0.5mm) 2nd layer: 0.15m washed pea gravel 3rd layer: 0.05m washed stone (20mm) 4th layer: 0.70m washed stone (40-50mm)	evenly perforated horizontal pipe across bed surface	short length, open ended collection pipe at base of wetland	*Phragmites communis* (4/m^2)	HLR= 8L/m^2	Secondary treatment (following septic tank/ settlement). Treating domestic wastewater.

Reference	Sizing	Depth	Media	Distribution	Collection	Vegetation	Loading Rates		Treatment type
Brix and Arias, 2005. 'the use of vertical flow constructed wetlands for on-site treatment of domestic wastewater: New Danish Guidelines'. Denmark	3.2 m^2/pe	1.4m	1st layer (top): 0.2m woodchip/ seashells (insulation) 2nd layer: 1.0m washed sand (d10= 0.25-1.2mm, d60=between 1-4mm) 3rd layer: 0.2m coarse gravel (8-16mm)	horizontal pipe layout. Pipes ø 32-45mm, hole ø 5-7mm every 0.4-0.7m.	pipe ø 70mm. Positioned in opposite direct to distribution pipes	*Phragmites australis* (4/m^2)	OLR= 19g BOD/pe/d	60l/ dose	Secondary treatment (following septic tank/ settlement). Treating domestic wastewater. 50% effluent recirculated.

Reference	Sizing	Depth	Media	Distribution	Collection	Vegetation	Loading Rates		Treatment type
United Nations Human Settlements Programme. 2008. 'constructed wetlands manual' Nepal.	0.8-1.5m^2/pe	0.70m	1st layer (top): 0.05m gravel (5-10mm) 2nd layer: 0.45m sand (0-4mm) 3rd layer: 0.05m gravel (5-10mm) 4th layer: 0.15m gravel (20-40mm)	horizontal pipes layout, evenly distributed downward facing inlets.	Slotted collection pipe in drainage layer of media	*Phragmites Karka, Phragmites Australis, Typha spp.* (4m^2/d)	OLR= 40g BOD/PE/d	HLR= 4L/m^2/dose (120L/m^2/d)	Secondary treatment (following septic tank/ settlement). Domestic wastewater
Onorm, 1997. Austria	4m^2/pe	0.95m	1st layer:0.50m sand (0.06-4mm) 2nd layer: 0.10m gravel (4-8mm) 3rd layer: 0.15m gravel (16-32mm)	Horizontal pipes, small holes ø 8mm every 0.75cm	Tile drains	*Phragmites australis*	20g COD/m^2/d		-

Reference	Sizing	Depth	Media	Distribution	Collection	Vegetation	Loading Rates		Treatment type
French Guidelines, 2005: technical recommendations for design. Treatment of domestic wastewater with filter plants. France	0.8m^2/pe	0.80m	1st layer (top):0.50m silica sand (d_{10}=0.25-0.4mm) 2nd layer: 0.15m fine gravel (5-10mm) 3rd layer: 0.15m coarse gravel (20-40mm)	vertical inlet points (one inlet per 5m^2). Pipe ø 60mm	Slotted collection pipe in drainage layer of media ø100mm	*Phragmites spp.*	25g COD/m^2/d	HLR= 4L/M^2/dose (120L/m^2/d)	Secondary treatment (following primary vfw)

Table 2-5 Typical tertiary influent characteristics on sites with tertiary treatment wetlands and sand filters, and their associated site consents. Measurements shown in mg/L. Mean (and range) shown for influent concentrations.

	Cooper et al., 1997		Schönerklee et al., 1997		Severn Trent Site 1*		Severn Trent Site 2**		Severn Trent Site 3***	
Pollutant	**Influent**	**Consents**	**Influent**	**Consents**	**Influent**	**Consents**	**Influent**	**Consents**	**Influent**	**Consents**
COD	-	-	27.7 (10.0-50.0)	-	50.9 (30.0-98.0)	N/A	53 (24.0-105.0)	N/A	71.3	N/A
TSS	17.7 (7.3-29.3)	30	-	-	30 .0 (15.0-58.0)	25	14.3 (8.3-30.2)	55	24.3 (3.0-52.0)	15
NH_4-N	6.4 (5.1-7.4)	10	1.1 (0.1-2.0)	-	6.2 (0.7-18.5)	3	2.1 (0.7-4.1)	N/A	11.4 (4.0-26.0)	3
TP	-	-	-	-	0.79 (0.23-1.80)	2	6.2 (1.9-9.7)	N/A	-	N/A
Ortho-P	-	-	1.3 (0.6-2.7)	-	0.37 (0.05-2.20)	N/A	-	N/A	-	N/A
BOD_5	18.1 (10.6-27.1)	20	(3-5)	-	8.2 (4.0-23.0)	10	7.7 (3.7-12.6)	30	10.3 (5.0-53.0)	10

*Site 1 – Primary settlement, Trickling Filter, Sand Filter

**Site 2 – Integrated Rotating Biological Contactor, Vertical Flow Wetland

***Site 3 – Integrated Rotating Biological Contactor, Horizontal Flow Wetland

2.8 References

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Chapter 3

Low energy tertiary treatment with vertical flow wetlands: a UK case study

3 Low energy tertiary treatment with vertical flow wetlands: a UK case study

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Abstract

A mature, gravel based vertical flow wetland (VFW) operating under tertiary application, was monitored for a 14 month period to identify the key environmental and operational factors and determine their influence on performance efficiency. The site was originally designed for a population equivalent of 115, but was operating to treat flow from seven domestic properties, and was therefore operating under capacity, with each of the two VFWs onsite receiving a hydraulic loading rate of 0.076m/d. As such, mean influent loadings onto the wetland were 1±0.4gTSS/m^2/d, 0.59±0.17gO_2/m^2/d and 0.16±0.07gNH_4-N/m^2/d for solids (TSS), 5-day carbonaceous biochemical oxygen demand ($cBOD_5$) and ammonium-nitrogen (NH_4-N), respectively. Results showed mean removal efficiencies of 47% for NH_4-N, 23% for TSS and 33% for $cBOD_5$, confirming the suitability for VFWs as a polishing technology even after 20 years of operation. Removal of total phosphorus and metals were negligible during the study and therefore the current VFW design would require modification to achieve future discharge consents. The key influential environmental and operational factors were determined as the influent pollutant loading and hydraulic loading rate.

3.1 Introduction

Eutrophication is the deterioration of freshwater ecosystems caused by entry of excessive nutrient levels into receiving watercourses, and is occurring on a global scale. Many sources contribute to this including the final effluent discharge from wastewater treatment sites. As a result, the need to produce better wastewater effluent quality has been identified and policies have been put in place with the intention of improving the water quality of all watercourses, primarily focusing on ecology within the systems and protecting drinking water resources (Water Framework Directive 2000/60/EC). With this as a driving factor it is anticipated that more stringent phosphorus, nitrogen species and heavy metal consents will be applied to wastewater treatment sites worldwide in the coming years, with the trend towards ammonia and phosphorus consents of ≤1mgNH_4-N/L and ≤0.5mgP/L, respectively, to bring into line with the existing river quality standards. This poses the largest threat of non-compliance to existing small wastewater treatment plants (WWTP) serving a population equivalent of 2000 or less (Griffin and Pamplin, 1998; Upton, 1995). In the UK, over 75% of all WWTPs are defined as small works, with typical consents ranging from 10mgBOD_5/L BOD, 15mgTSS/L suspended solids and 5mgNH_4-N/L ammonia-nitrogen ("10/15/5" standards as 95th percentile) to 40mg/L for BOD and 60mg/L for suspended solids ("40/60" standards as 95th percentile), with the tightest consents applied to sites discharging to sensitive watercourses or to sites of special scientific interest (DEFRA, 2002; Johnson et al., 2007). Whilst small WWTP have historically been designed to remove organic matter and solids, there is a clear move towards ammonia removal in the short term, and phosphorus and metals in the longer term. A potential solution to achieving these projected consents is with the addition of a tertiary treatment process, such as constructed wetlands.

In the UK, application of tertiary constructed wetlands has been limited to horizontal flow systems (HF), originally designed to remove particulate organic matter and solids, and can run without any energy input and with little head loss, making them easy to integrate into existing flowsheets. However, the nitrification potential within these systems is limited due to the predominately anaerobic and anoxic environments created within the permanently saturated bed, such that it is unlikely they will be able to contribute significantly to the removal of ammonia without modifications to their design and operation. Potential approaches to increase the oxygen transfer potential needed for nitrification, has seen the progression of artificially and passively aerated technologies in the form of forced aeration horizontal flow wetlands (Butterworth et al., 2013) tidal flow wetlands (Wu et al., 2011) and vertical flow wetlands (Besançon et al., 2017).

Vertical flow wetlands (VFWs) are passively aerated systems, commonly applied for whole or secondary wastewater treatment, particularly within Europe (Molle et al., 2005). The body of these VFWs typically consist of an arrangement of a deep sand layer as the main treatment, a transition layer of pea gravel and a drainage layer of large gravel or cobles. The media type and configuration promotes physical entrapment of solid material and provides a large surface area for biofilm adhesion and establishment, enhancing biological wastewater treatment through increased water to biofilm contact time (Brix and Arias, 2005b; Molle et al., 2005). Typical operation of these systems involve an intermittent feeding regime, creating a series of daily feed and rest cycles, promoting oxygen transfer by convection into the bed and then by diffusion into the biofilm.

Vertical flow wetlands have been applied for whole and secondary treatment of wastewater for over 40 years worldwide (Seidel, 1976), and for secondary treatment for over 20 years in the UK (for industrial and private treatment systems), and successfully remove solids, organic and ammonia from influent wastewater, with reported efficiencies of >80% suspended solids removal; >90% chemical oxygen demand removal; and >75% of ammonium-nitrogen removal (Brix and Arias, 2005a; Weedon, 2010). Although VFWs have been established as a nitrifying technology, their use within tertiary or polishing applications is somewhat outdated, and limited to immature systems (Cooper et al., 1997; Schönerklee et al., 1997), with newer applications yet to be quantified.

UK guidelines for reed bed design, construction and maintenance (Griggs and Grant, 2000a), are somewhat different to other VFW guidelines in that gravel is recommended as the main treatment layer, although a shallow sand layer is recommended on the surface to reduce solids infiltration whilst providing an even distribution to the flow. Comparison of ammonia removal for gravel and sand based systems, treating primary treated effluent, reveals removal efficiencies of 56% (Korkusuz et al., 2005) and ≥75% (Brix and Arias, 2005a) respectively. The observations are congruent with the reduced hydraulic conductivity and increased contact surface area associated with the small sand media. However, such attributes also influences the degree of susceptibility to clogging if the systems are hydraulically or organically overloaded (Sani et al., 2013; Winter and Goetz, 2003). The consequence is an increase in maintenance which may challenge the preferred refurbishment frequency of between eight (Griffin et al., 2008) and 15 years (Cooper et al., 1996). Considerations for the use of vertical flow wetlands for tertiary applications should therefore account for such balances in light of the much lower organic and solids load that are expected to be present compared to secondary systems.

The aim of this study was to determine the efficacy of a mature, gravel based VFW applied for the tertiary treatment of domestic wastewater. This was achieved through a 14 month monitoring period, providing a knowledge base of key environmental and operational factors that influence the performance of the wetland within this application, and the implications of gravel based designs for reaching future consents.

3.2 Materials and Methods

3.2.1 Site Description

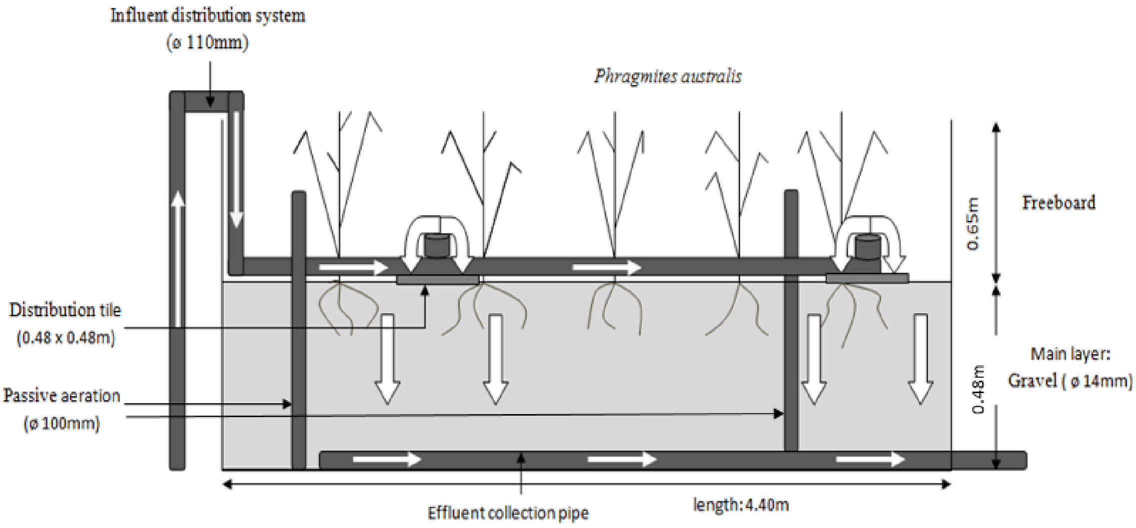
The study was conducted on a small sewage treatment works originally designed to treat a population equivalent of approximately 115, but now serving only seven properties under a separate sewer system (i.e., there are no storm flows in the sewage). The site comprised an integral rotating biological contactor (RBC) followed by two vertical flow wetlands. The integral RBC houses a primary settling tank, two biozones and a final settling tank. Once treated, the RBC effluent was retained within an external holding tank from which it is intermittently pumped onto two parallel, above ground, tertiary VFWs, each with an area of 9.5m^2

(Figure 3-1). The feed pump to the wetland was operated on a level switch within the holding tank, therefore feed cycles were not flow dependant. The wetlands were originally installed on the site in 1992, but underwent pipe repairs, surface sludge removal, and reed harvesting in November 2011.

Each of the wetlands had an inlet distribution system consisting of a single 110mm diameter PVC pipe, which was centralised and extended over the length of the bed, and contained two evenly distributed inlet points with a 110mm diameter opening. The flow of wastewater onto the wetlands was dispersed using concrete distribution tiles, each with a size of 0.48m by 0.48m, which are positioned under each of the inlet points. The wetland media primarily consisted of single sized, 14mm gravel and had a depth of approximately 0.48m. Each of the wetlands were planted with *Phragmites austalis* at 4 seedlings/m^2. Flow was recorded every minute with a levelogger (levelogger EDGE model 3001, Solinst Canada LTD) which was positioned within the effluent collection chamber. The mean flow to each wetland was 0.72m^3/d, equating to a mean hydraulic loading rate of 0.076m/d. Wetland influent loadings recorded between August 2012 and October 2013 were 1±0.4gTSS/m^2/d for total suspended solids (TSS), 4±1.6g/m^2/d for chemical oxygen demand (COD) and 0.16±0.07gNH_4-N/m^2/d for ammonium-nitrogen. The treatment works, being a very small site and discharging to a robust watercourse, has only two stated consents; 55mgTSS/L for the suspended solids and 30mg/L for $cBOD_5$, based on 95th percentiles. Surface sludge accumulation measurements were performed by inserting a metal ruler into the sludge layer and recording the height, which was conducted on a monthly basis from January 2013 onwards.

A

B



C

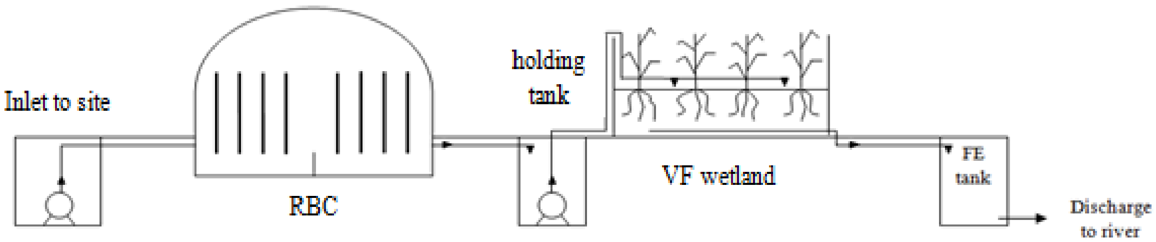


Figure 3-1 (A) Photo of the vertical flow wetland after reed harvesting in October 2011. (B) A schematic of the study vertical flow wetland, and (C) Process flow of the trial site.

3.2.2 Analytical Methods

Duplicate grab samples were collected from the wetland inlet and from within the final effluent sampling chamber, once a week for the initial three months of the study and once fortnightly thereafter, for a total period of fourteen months between August 2012 and September 2013 (*n*= 60). Wastewater samples were collected in 1L plastic bottles and were transported to the laboratory for same day analysis.

The samples were analysed for ammonium-nitrogen (NH_4-N), nitrite-nitrogen (NO_2-N), nitrate-nitrogen (NO_3-N), total (t) and soluble (s) chemical oxygen demand (COD), and total phosphorus (TP) using commercially available standardised test kits (Hach Lange, Manchester, UK) according to the manufacturer's protocol, and determined using spectrophotometry (DR2800, Hach Lange, Manchester, UK). Standard Methods (American Public Health Association (APHA), 2005) were used to determine the total and volatile suspended solids (TSS and VSS, respectively) within the wastewater samples, using a three piece filtration apparatus and glass microfiber filter papers of 70mm diameter and a particle retention size of 1.2µm. The 5 day carbonaceous biochemical oxygen demand ($cBOD_5$) was determined according to Standard Methods (American Public Health Association (APHA), 2005). The BOD dilution water was prepared using BOD nutrient buffer pillows (Hach Lange, Manchester, UK) following the manufacturer's protocol, and nitrification inhibitor was added to each of the BOD samples. Samples for BOD were used undiluted as the values were typically close to the detection limit of the method. Soluble COD samples were filtered using a 0.45µm filter and the COD in the filtrate was determined.

Metals analysis were conducted on each of the wetland influent and effluent samples collected between December 2012 and September 2013. Where same day analysis was not possible 30ml of each wastewater sample was filtered through a 0.45µm filter, preserved in 1.5ml of concentrated trace metal grade nitric acid and stored at 4°C until analysis was possible. Samples were analysed to determine concentrations of nickel (Ni), lead (Pb), copper (Cu), chromium (Cr), cadmium (Cd) and zinc (Zn) using inductively coupled plasma- mass spectrometry.

Onsite analysis of sludge accumulation and in-bed dissolved oxygen were conducted following a flood event that occurred in November 2011. Sludge accumulation measurements from the wetland surface were obtained using a ruler and were recorded on a monthly basis following complete surface drainage in January. Dissolved oxygen levels from within the wetland were obtained during a feeding cycle using a LDO sensor probe and a portable multi-meter (HQ40d multi-meter; Hach, Germany), with the probe positioned within the preinstalled passive aeration pipework.

3.2.3 Statistical Analysis

Statistical analyses were conducted using the STATISTICA v14 software package (StatSoft Inc, Oklahoma, USA). To determine the significance between the parameters measured during each of the seasons, a step wise multiple regression test was conducted, and the residual distribution checked after the tests. Correlations between flow rate, drainage time, sludge accumulation and rainfall were also analysed.

3.3 Results

The raw wastewater to the inlet of the wastewater treatment works had contaminant concentrations of approximately half of the average low-strength untreated domestic sewage (Tchobanoglous et al., 2004), with influent concentrations of 74mgTSS/L for suspended solids, 74mgCOD/L for total chemical oxygen demand and 6.45mgNH_4-N/L for ammonium-Nitrogen. The RBC, although oversized in this instance for the load received, showed concentration removals of approximately 80% of TSS, 30% of COD and 70% of NH_4-N. This suggests that the RBC is performing above average for ammonia removal (an average of 20% reduction), (Tawfik et al., 2006), below average for COD removal (an average of 77-86% reduction), (Tawfik et al., 2006), and is in agreement with documented TSS removal (an average of 77% removal), (Hanhan et al., 2005). As the 95th percentile of the overall RBC effluent concentrations are 23mgTSS/L for suspended solids and 11mg/L for $cBOD_{5,}$ it suggests that the RBC treatment alone achieves within the consented parameters for the site, and confirms that the purpose of the wetlands onsite are to provide a polishing treatment.

The average final effluent concentrations of the site, measured at the outlet of the wetlands, were 11±5mgTSS/L for suspended solids and 5±2mgO_2/L for $cBOD_5$ (Table 3-1). Maximum final effluent concentrations of 26mgTSS/L for suspended solids and 9mgO_2/L for total suspended solids were recorded during the study period, which were still well below the site environmental discharge permit. In spite of a lack of regulatory nutrient removal requirements, average NH_4-N final effluent concentrations of 1.11±0.66mgNH_4-N/L were achieved.

Flow rates into the treatment works were significantly higher in autumn than in all other seasons, with the recorded rainfall data showing not to be an influential factor (Table 3-2). This was expected, as the site is fed by a separate sewer with limited infiltration. More feeding batches were recorded in winter, in spite of higher flows being observed in autumn, due to inconsistency of flow during a feed cycle. The drainage time of the wetlands was unaffected by flow rates, averaging 6 minutes regardless of season. This was expected of unsaturated gravel beds, where clean and clogged gravel have a permeability of 109.8cm/min (Lewis et al., 2006) and 0.2cm/min (Molle et al., 2006) respectively. Indeed, the most influential factor contributing to drainage time is the level of surface sludge. Multiple regression analysis showed that whilst the relationship between the performance and key operational parameters changed depending on the season, the hydraulic and specific pollutant load were the main factors affecting the wetlands removal rate (Table 3-3).

Table 3-1 Seasonal and overall mean and standard deviation of the VFW influent and effluent concentrations across the seasons between August 2012 and September 2013.

	TSS	VSS	tCOD	sCOD	$cBOD_5$	NO_2-N	NO_3-N	NH_4-N	TP
Autumn									
VFW Influent (mg/L)	14 ± 4	11 ± 3	43 ± 18	18 ± 12	8 ± 0.1	0.5 ± 0.2	6.5 ± 2.3	1.7 ± 0.8	5.5 ± 1.9
VFW Effluent (mg/L)	9 ± 3	7 ± 3	34 ± 15	17 ± 11	8 ± 0.3	0.3 ± 0.2	7.0 ± 2.3	1.0 ± 0.7	5.4 ± 2.0
Winter									
VFW Influent (mg/L)	13 ± 3	9 ± 3	35 ± 7	23 ± 9	7 ± 2	0.4 ± 0.1	10.0 ± 2.5	1.1 ± 0.2	3.3 ± 0.9
VFW Effluent (mg/L)	7 ± 3	6 ± 3	27 ± 8	17 ± 5	6 ± 4	0.2 ± 0.0	10.9 ± 2.7	0.3 ± 0.1	3.2 ± 0.9
Spring									
VFW Influent (mg/L)	19 ± 10	16 ± 9	68 ± 21	47 ± 5	7 ± 1	0.8 ± 0.3	8.5 ± 2.0	2.3 ± 0.5	7.7 ± 1.2
VFW Effluent (mg/L)	16 ± 7	13 ± 7	61 ± 18	42 ± 7	4 ± 0.3	0.4 ± 0.2	9.9 ± 1.8	1.1 ± 0.4	7.7 ± 1.2
Summer									
VFW Influent (mg/L)	14 ± 4	13 ± 4	62 ± 17	45 ± 8	10± 2	0.5 ± 0.3	4.8 ± 1.2	3.1 ± 0.6	8.4 ± 1.0
VFW Effluent (mg/L)	13 ± 6	12 ± 5	53 ± 17	40 ± 7	5 ± 1	0.3 ± 0.2	6.2 ± 1.6	1.9 ± 0.7	8.4 ± 1.0
Overall data									
VFW Influent (mg/L)	14± 5	12 ± 5	53 ± 19	35± 15	8 ± 2	0.6 ± 0.3	7.0 ± 2.7	2.1 ± 1.0	6.2 ± 2.3
VFW Effluent (mg/L)	11 ± 5	9 ± 5	46 ± 18	30 ± 14	5 ± 2	0.3 ± 0.2	7.9 ± 2.8	1.1 ± 0.7	6.2 ± 2.4

Table 3-2 Seasonal variation in key VFW operational parameters.

		Flow rates (m^3/d)		Water temperature (°C)		Rainfall (mm/d)		number of batches		drainage time (mins)	
Season	Sample size	range	average (±SD)	range	average (±SD)	range	average (±SD)	range	average (±SD)	range	average (±SD)
Autumn	20	0.02-33.9	3 ± 5	3.5-18.6	14 ± 2	0-30	3 ± 5	8-410	91 ± 88	3-13	6 ± 3
Winter	10	0.00-16.8	1 ± 3	7.1-10.5	9 ± 1	0-23	3 ± 4	2-343	118 ± 80	1-8	4 ± 2
Spring	12	0.01-13.5	1 ± 2	6.9-14.1	10 ± 2	0-24	4 ± 5	8-260	72 ± 43	2-8	5 ± 2
Summer	18	0.00-24.6	1 ± 3	13.5-20.9	18 ± 2	0-28	1 ± 4	0-233	37 ± 36	1-8	5 ± 2

Table 3-3 Multiple regression relationship between the VFW removal efficiencies of NH4-N, TSS and COD and the key operational parameters within the seasons and for the overall study.

	Equation	R^2	P value	Significant
Autumn				
NH_4-N removal rate	0.01 + (0.225* NH_4-N load) + (0.260*HLR)	96	<0.05	Yes
TSS removal rate	-0.231 + (0.266*TSS loading)	53	<0.05	Yes
COD removal rate	1.726 + (0.383*COD loading) - (0.152*temperature)	89	<0.05	Yes
Winter				
NH_4-N removal rate	-0.022 + (0.992*NH_4-N load) - (0.013*HLR) + (0.0001*number of feeds) - (0.0005*rainfall) + (0.002*temperature)	99	<0.05	Yes
TSS removal rate	-0.280 + (0.023*TSS concentration)	86	<0.05	Yes
COD removal rate	1.282 - (0.137*temperature)	47	<0.05	Yes
Spring				
NH_4-N removal rate	-0.017 + (0.331*NH_4-N load) + (0.008*NH_4-N concentration)	99	<0.05	Yes
TSS removal rate	0.649 + (1.566*TSS load) - (1.345*HLR) - (0.040*TSS concentration)	95	<0.05	Yes
COD removal rate	-0.078 + (1.331*COD load) - (2.663*HLR)	72	<0.05	Yes
Summer				
NH_4-N removal rate	0.007 + (1.118*HLR)	97	<0.05	Yes
TSS removal rate	0.001 + (0.006*rainfall)	14	>0.05	No
COD removal rate	-0.008 + (0.020*rainfall)	77	<0.05	Yes
Overall data				
NH_4-N removal rate	0.004 + (0.245* NH_4-N load) + (0.223*HLR)	97	<0.05	Yes
TSS removal rate	-0.002 + (0.357*TSS load)	68	<0.05	Yes
COD removal rate	0.256 + (0.418* COD load) - (9.377* HLR) - (0.012*COD concentration)	89	<0.05	Yes

3.3.1 Solids Removal in tertiary vertical flow wetlands

The mean TSS areal removal rate within the VFW was 0.3g/m^2/d, with a mean VSS removal rate of 0.2g/m^2/d. The overall mean VSS accounted for approximately 87% of the influent TSS, and approximately 85% of the effluent, showing no change between inlet and outlet solids composition.

The greatest seasonal variation in TSS mass loading was seen between autumn, receiving the highest inlet load at 2g/m^2/d, and summer with the lowest inlet load at 0.4g/m^2/d, corresponding to the highest (0.7g/m^2/d) and insignificant removal rates, respectively (Figure 3-2). The statistical analysis confirmed TSS loading to be the main factor influencing the removal rate of TSS by the wetlands (Table 3-3), rather than influent concentrations. In agreement with other vertical flow wetland findings, the highest TSS loading rates onto the wetland correspond to the highest TSS outlet concentrations (Kadlec and Wallace, 2009).

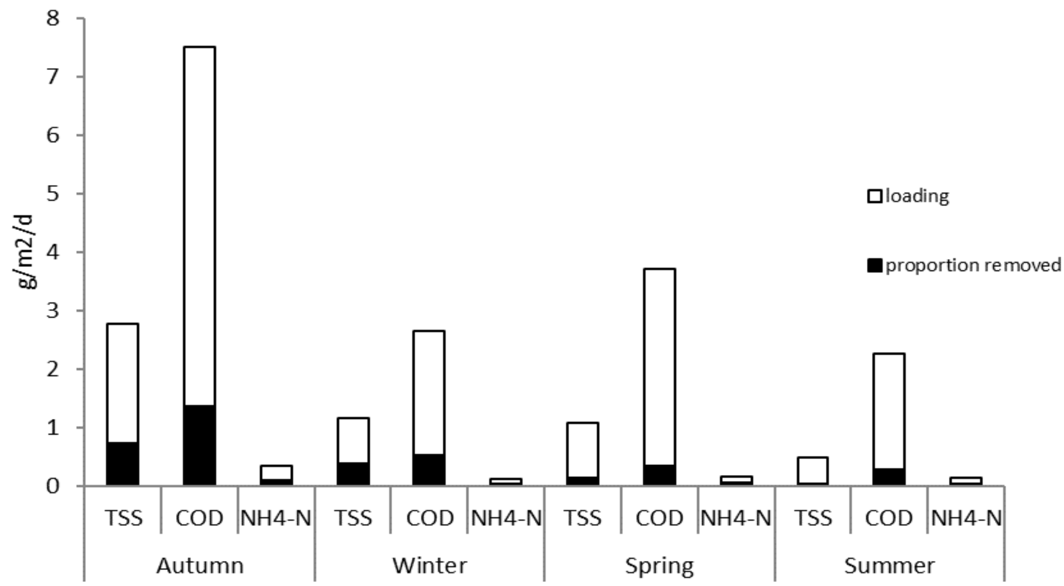


Figure 3-2 Seasonal variations in TSS, COD and NH_4-N loading and areal removal rate within the tertiary VFW. The whole column, containing both shaded and non shaded portions, represent the loading and the shaded area represents the proportion of loading that was removed during treatment.

3.3.2 Organics Removal in Tertiary Vertical Flow Wetlands

The mean total COD mass loadings of the VFWs were reduced from 4±2g/m^2/d to 3±1g/m^2/d during treatment. Soluble COD accounted for approximately 68% of the mean influent total COD, and 73% of the mean effluent total COD, with an areal removal rate of 0.4g/m^2/d. The overall mean $cBOD_5$ removal rate was 0.2g/m^2/d, resulting in a mean effluent $cBOD_5$ concentration of 5±2mg/L (Table 3-1).

The greatest seasonal difference in the organic loading rate occurred between autumn and summer: the VFW COD loading in autumn was recorded as 6gCOD/m^2/d, which corresponds to the highest removal rate (1g/m^2/d), and the lowest seasonal VFW organic loading rate recorded for summer (2gCOD/m^2/d), corresponding to the lowest removal rate (0.3g/m^2/d). The VFW inlet concentrations for the total COD were significantly higher during the spring and summer and corresponded with the lowest removal efficiencies. The statistical analysis highlighted, again, the importance of loading over influent concentrations on removal rates.

The removal of soluble COD were somewhat contradictory to that of the total COD. Autumn also saw the highest organic loading rate of sCOD (3±2g/m^2/d), as with the tCOD, however it provided the lowest removal rate and the lowest removal efficiency. The lowest seasonal sCOD loading occurred over the duration of the winter period and corresponded the greatest removal rate.

3.3.3 Nutrient Removal in tertiary Vertical Flow Wetlands

The removal rate for NH_4-N was low, averaging 0.07g/m^2/d. This is likely the result from low influent NH_4-N concentrations, which were consistently below 4mg/L (Table 3-1). Ammonium-nitrogen removal was dependent on NH_4-N loading and hydraulic load. Concomitant with NH_4-N reduction rates, nitrate concentrations increased by 12% of the inlet concentration during treatment, whilst the nitrite concentrations decreased from 0.56±0.28mgNO_2-N/L to 0.32±0.18mgNO_2-N/L, indicating that nitrification is taking place within the wetland. Seasonal data obtained for both NO_2-N and NO_3-N indicate the lowest nitrification capacity was during autumn (1% and 5% respectively). It is assumed that denitrification potential within the study wetland is limited, due to the degree of oxygenation within the bed (Cooper, 2005) thus providing inhospitable conditions for adequate denitrification microbial activity. A nitrogen mass balance in each season confirms nitrification as the main ammonia removing process, with less than 1% of the influent nitrogen unaccounted for in the effluent and potentially being emitted as nitrogen or nitrous oxide gas (Fuchs et al., 2011) (Table 3-4), suggesting denitrification may be occurring in small anaerobic pockets within the wetland media. The influent to the site (prior to RBC treatment) contained concentrations of 0.68mgNO_2-N/L for nitrite-nitrogen; 3.88mgNO_3-N/L for nitrate-nitrogen, with both the RBC and VFW appearing to sufficiently nitrify as decreasing NH_4-N concentrations during treatment correspond to increasing NO_3-N concentrations.

Table 3-4 Basic nitrogen species mass balance of the influent and effluent VFW loadings across the seasons (g/d).

	Autumn		Winter		Spring		Summer	
	Influent	Effluent	Influent	Effluent	Influent	Effluent	Influent	Effluent
NH_4-N	3.61	2.13	1.21	0.35	2.13	1.07	1.88	1.15
NO_3-N	14.10	15.10	11.36	12.37	7.96	9.30	2.96	3.77
NO_2-N	1.17	0.73	0.42	0.20	0.74	0.37	0.33	0.21
Total	18.88	17.96	12.99	12.92	10.83	10.74	5.17	5.13

Average total phosphorus influent concentrations ranged between 3.3±0.9mg/L (winter) and 8.4±1.0mg/L (summer). This significant change in concentrations was unexpected and could be potentially attributed to farming practices in the warmer months within the catchment of the served households. Regardless of the source and influent concentration, there was no statistically significant removal in TP in the treatment wetland. This was expected of a mature gravel bed system, as is the case in other gravel systems (Korkusuz et al., 2005) and even sand based systems (Paing and Voisin, 2005; Torrens et al., 2009).

3.3.4 Metals Analysis in Tertiary Vertical Flow Wetlands

The analysis of cadmium, lead and nickel were below the limit of detection (LOD= 1µg/L) for influent and effluent samples. The overall data suggests an increase in zinc effluent concentrations from 26±15µg/L in the influent to 36±20µg/L in the effluent. There was no significant difference ($p>0.05$) between overall influent and effluent concentrations of copper and chromium. Variations within the seasonal influent Cr, Cu and Zn concentrations were observed (Figure 3-3). Both Zn and Cu show similar variations between the seasons, being lower during autumn and winter and higher during spring and summer (Figure 3-3, A & B), in contrast to trends shown for Cr with higher concentrations during autumn and winter than spring and summer (Figure 3-3, C).

3.3.5 Vertical Flow Wetland Surface Sludge and Onsite Analysis

During November, approximately 15cm of flooding on the VFW surface was observed. This resulted from a higher than average monthly rainfall of 120mm, which exceeded the seasonal average by 40mm. The wetland surface water levels gradually decreased through December, until complete drainage was achieved during early January 2013 (Figure 3-4). It was at this time that an increased level of surface sludge was observed over the wetland, comprising a depth of 45mm, and so monthly sludge accumulation measurements were recorded there forward. During this time a short survey of in-bed dissolved oxygen readings (n=4) showed levels of between 2.7 and 4.3mgO_2/L.

A strong correlation was observed between sludge accumulation and temperature (R^2=0.77) and sludge accumulation and the number of feeds (R^2=0.77). Weak correlations were observed between monthly sludge levels and rainfall (R^2=0.11) and between sludge levels and flow (R^2=0.12).

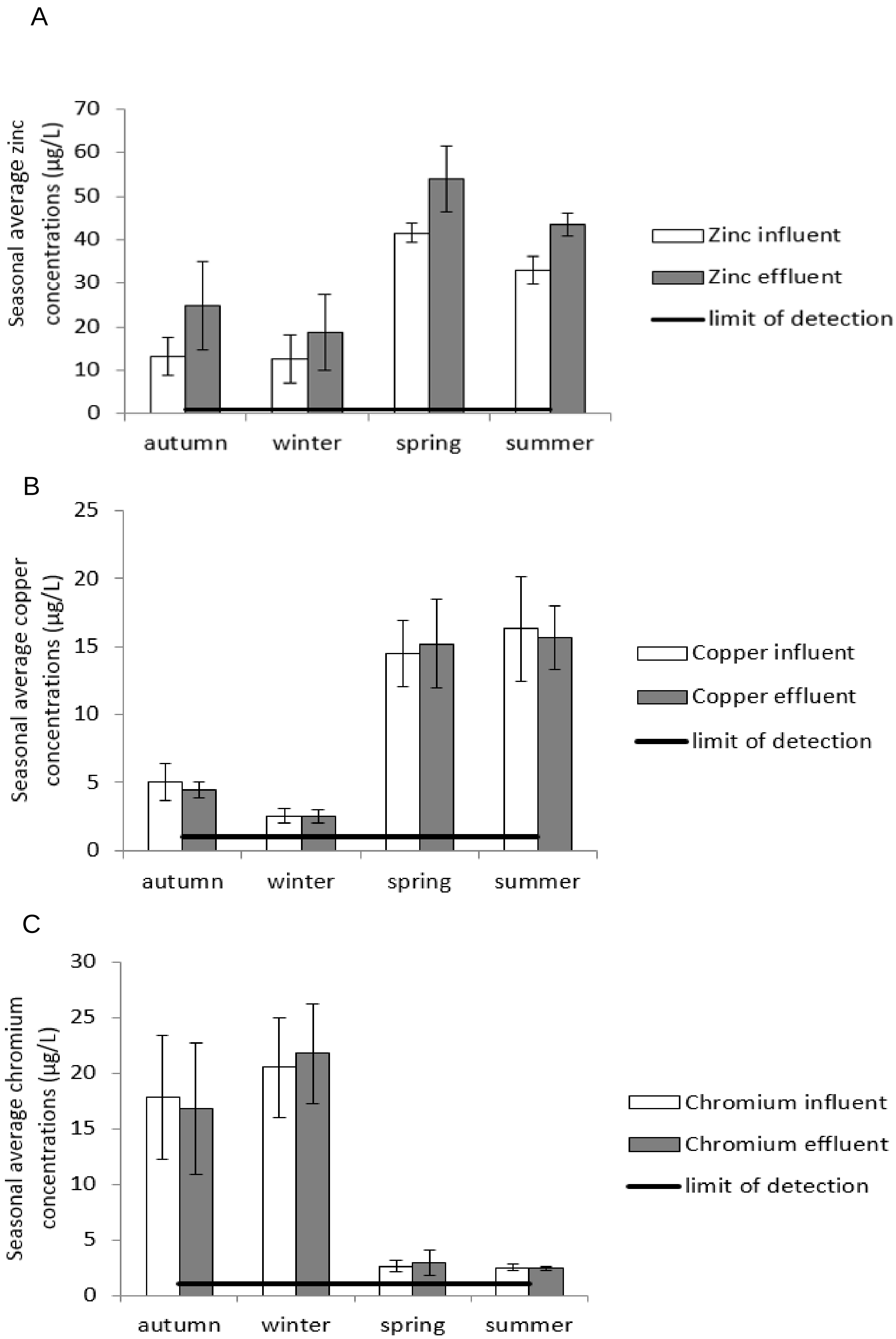


Figure 3-3 Seasonal variation between influent and effluent concentrations of (A) Zinc, (B) Copper and (C) Chromium. The limit of detection (1gµ/L) is included on the graph. The error bars show the standard deviation.

A

B

C

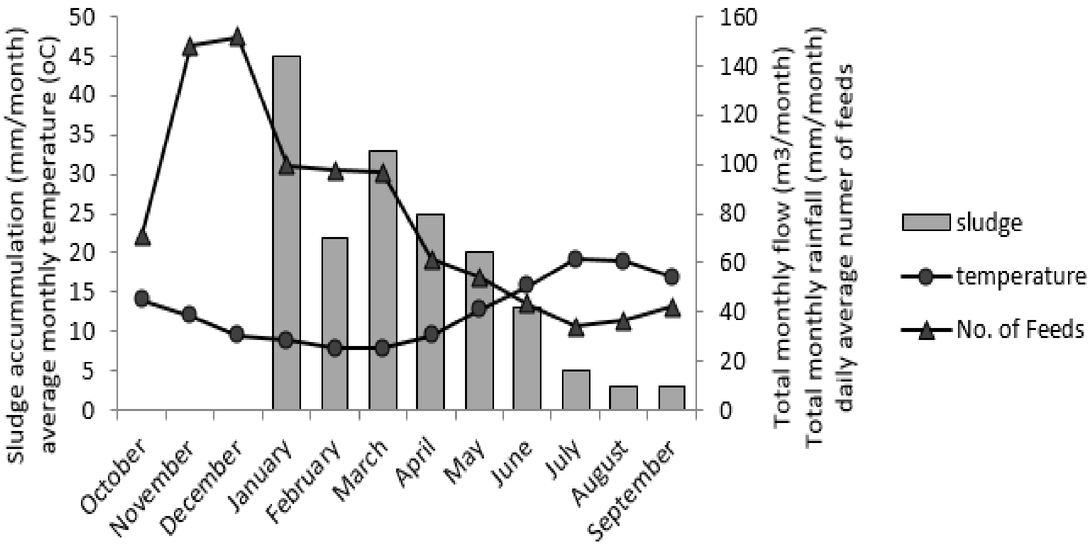


Figure 3-4 (A) Photo of the sludge accumulation observed in January 2013, (B) Photo of the wetland sludge accumulation observed in September 2013, (C) Stacked data to show the trends between monthly sludge accumulation, average daily temperature for each month and average number of intermittent feeds to the wetlands.

3.4 Discussion

This study monitored the treatment and operational behaviours of a mature gravel based full-scale tertiary vertical flow wetland treating domestic wastewater. The wetland system has been operating for 20 years with a refurbishment one year prior to the monitoring period. Overall the study demonstrates the ability of such tertiary wetland systems to provide polishing of RBC effluents by reducing concentrations of ammonium-nitrogen, total suspended solids and organic material.

The process of nitrification is known to be susceptible to lower temperatures, due to reduced microbial activity (Vymazal, 2007) and is believed to perform at an optimum ammonia removal efficiency at temperatures of between 15°C and 25°C (Tsihrintzis et al., 2007). Water temperatures recorded during this study ranged between 7-19°C, yet statistical analysis determined that temperature did not significantly influence the nitrification capacity within the wetlands ($p>0.05$) for the overall data set, but did show to be an influencing factor during the winter months. This is in agreement with other vertical flow wetland studies, which have suggested that temperature does not play a significant role in nitrification capacity. Furthermore it is reported that successful nitrification can be performed at temperatures of down to 4°C in Europe (Brix and Arias, 2005a; Prochaska et al., 2007) and 5°C in the UK, with normal nitrification functionality returning at temperatures of 8°C and above (UKWIR, 2011). Analysis of the data revealed the most influential factors in the NH_4-N removal rate, across all the seasons, to be the NH_4-N loading and the hydraulic loading rate (Table 3-3). The greatest areal removal rate, observed during the autumn, 0.097g/m^2/d, corresponded to the highest influent NH_4-N loading, suggesting that the wetland was operating well below its maximum capacity. This concurs with findings from young tertiary sand-based vertical flow wetland investigations, from within the initial three years of operation (Cooper, Smith and Maynard, 1997) and young gravel-based horizontal flow wetlands, from within the initial 9 months of operation (Butterworth et al., 2013) offering confidence in their potential use for tertiary nitrification. It is interesting to note that site influent (prior to RBC treatment) nitrate and nitrite concentrations, although not significant, were greater when compared to the influent concentrations of vertical flow wetland systems, namely secondary systems (Brix and Arias, 2005b), and tertiary horizontal flow studies (Butterworth et al., 2013).

In secondary sand based VF applications, the limiting factor is related to oxygen transfer and, ultimately, the ability to maintain sufficient dissolved oxygen for the biofilm to fully process the adsorbed nutrients. Operationally, the feed frequency is linked to performance whereby increasing the feed frequency generates a commensurate increase in the hydraulic retention time (Bancolé et al., 2003) but reduced oxygen delivery can increase solids accumulation in the upper layers (Torrens et al., 2009) and as such lower feed cycles are generally preferred (Molle et al., 2006). In the current study, higher nitrification performance (71%) was observed during winter, where the highest number of feed cycles (116/day) were recorded, supporting the suggestion of the system being load limited such that future systems should be operated with a high feed frequency. This is further supported through a short survey of dissolved oxygen, which showed average within-bed concentrations of 3mg/L, hence above the required level.

Surface sludge levels in winter were the highest of all values recorded, which corresponded with the highest removal efficiency for TSS. Notably, this is where the correlation with TSS load was weakest (Table 3-2). The effect of the sludge layer on TSS is used to its advantage within some whole treatment vertical flow wetland designs. For instance, in French-type first stage vertical flow wetlands, the influent suspended solids form a blanket sludge layer over the gravel surface, aiding in an even flow distribution at a reduced velocity (Chazarenc and Merlin, 2005; Molle et al., 2005; Paing and Voisin, 2005). This principle works well within a feed/rest rotation arrangement, in which the wetlands receive intermittent feeding cycles and subsequent resting periods at a 1:2 ratio, and obtain removal efficiencies of between 86-99% of TSS on first stage, whole treatment beds. In this study, the performance during the “flooded” period was shown to have the greatest removal efficiencies, suggesting a similar beneficial effect of the surface sludge layer although this could also reflect the impact of increased load on a load limited system. Strong correlations between sludge accumulation and temperature and between sludge accumulation and number of feeds support the theory that resting periods enable surface layer mineralisation and sludge volume reduction. This is common in French type systems (Molle et al., 2005) and compact sand based vertical flow wetland systems (Weedon, 2010).

Metal removal during wastewater treatment processes is typically performed through adsorption (or ion-exchange), with the use of specialised filtration material, or through precipitation mechanisms aided by the addition of hydroxides or sulphides, none of which were used within the study site, therefore limited metal removal was expected. This was the case with five out of six metals analysed (Cr, Cd, Pb, Ni and Cu), however zinc analysis revealed a 27% overall increase between inlet and outlet concentration. The highest increase between inflow and outflow zinc concentrations was observed during autumn (47%) and winter (32%). Previous research into the effect of metal uptake by wetland plants have shown heavy metals to predominantly accumulate within the roots and underground portion of Phragmites, however, zinc has been shown to also accumulate within the leaves and stems, particularly in October (Weis and Weis, 2004). It is possible that the lack of annual wetland plant harvesting at the study site led to detritus build up on the wetland surface and potential leaching of zinc into the wetland during autumn and winter months. Additionally, previous sand based vertical flow wetland studies have shown that metals accumulate within the sludge sediments and can be released from a solid to a soluble phase as a result of sludge erosion, resulting from disturbances due to high flow and rainfall (Lee and Scholz, 2007; Marchand et al., 2010). Hydraulic loading rates during this study were highest during autumn and winter, corresponding to the highest release of zinc, suggesting that zinc may have been released as a result of sludge sediment disturbance.

Overall the results here have demonstrated the feasibility of vertical flow wetlands to operate as tertiary polishing processes especially in relation to reduction in ammonia, organics and solids. Limited removal of metals and dissolved phosphorus is expected once the capacity of the media has been reached and so removal only during the first year of operation is likely unless higher capacity media are used (Arias and Brix, 2005b). Although, if sludge accumulation occurs, consideration needs to be taken with reference to a risk of occasion metal flushes leaching out of the sediment.

Going forward, media type and size should be a key area of consideration when designing a VFW for wastewater treatment and appropriate media should be selected based on the proposed application and level of treatment required. Current UK guidelines for secondary VFWs propose the use of gravel with an additional sand layer on top if enhanced treatment is required (Griggs and Grant, 2000a, 2000b), whereas in other parts of Europe, sand is used as the main treatment media (Arias and Brix, 2005a; Molle et al., 2005). This difference in design for similar application is to account for the difference in the level of treatment required and for the intended operating philosophy. In the UK, single stage compact VFWs operated without resting periods are the preferred option, due to the limited land use required for this set up. However, in Europe, where additional land acquirement is more likely, VFWs are operated on a rotational basis or with recirculating flow for enhanced treatment with lower flow throughputs. Therefore, for the optimal VFW design a balance is required between treatment capacity and hydraulic control for any given system. In the case of tertiary treatment, the relatively low concentrations of solids, organics and ammonia expected in the VFW feed suggest the benefits of sand, for enhanced treatment and rotation, for increased aeration and longevity due to reduced clogging risk, may be less significant than in secondary applications as demonstrated in the current tertiary treatment case study.

3.5 Conclusions

An assessment of the performance, and the associated influential factors, of a full-scale mature gravel-based vertical flow wetland for tertiary wastewater treatment was conducted to determine contaminant removal and operational behaviours. Observations during the study have demonstrated the use of gravel based vertical flow wetlands, even after 20 years of operation, to provide polishing of solids, organics and ammonium-nitrogen within secondary effluents, however, removal of TSS and NH_4-N was not deemed statistically significant during the overall study, therefore the null hypothesis was accepted. In this instance the final effluent quality surpasses the current site discharge consents and have been achieved to within the tighter consented 10/15/5 standard limits applied to sites discharging to sensitive watercourses. However, this is due to low influent concentrations rather than enhanced treatment. Data showed a greater nitrification rate was achieved when greater NH_4-N loads were applied, indicating further treatment capacity could be achieved with increased loadings. However removal of phosphorous and metals during wetland treatment was negligible, suggesting the current system will be unable to comply to future proposed consents without modification or adjustments to the current design. There was a clear seasonal influence on performance efficiencies, however, statistical analysis revealed the most influential factors to be influent pollutant loading and the hydraulic loading rate. Temperature and rainfall, the factors most linked to differences between the seasons, had no significant influence on removal performances when considering the data for the full trial, however when broken down into seasons temperature influenced COD removal during autumn, rain and temperature influenced NH_4-N and COD removal during winter and rain influenced TSS and COD removal during the summer months.

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Chapter 4

Impact of loading rate on the operation and performance efficiency of pilot scale tertiary vertical flow wetlands

4 Impact of loading rate on the operation and performance efficiency of pilot scale tertiary vertical flow wetlands

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Abstract

Media clogging is prevalent in vertical flow wetlands (VFWs) and can significantly impact performance potential by reducing the oxygen transfer to within the bed, and ultimately reduces the longevity of the system. Through optimisation, the operational strategies applied to VFWs can be adapted to prevent potential clogging events from occurring and can assist in the remediation of previously clogged systems. The hydraulic loading rate (HLR) is considered a main contributor to clogging occurrences due to associated solids and organic loads, as such its optimisation is fundamental to successful operation of the system. This study used eight unplanted large scale pilot plants to determine the impact of HLR on the performance and clogging potential within VFWs under tertiary application. The study was conducted over two stages, the first to determine the hydraulic load capacity, degree of clogging and treatment potential using HLRs of between 0.05 and 1.04m/d. The second stage utilised the four most promising HLRs determined during first stage trials for continued performance analysis. Results show HLRs of ≤0.2m/d, equating to a solids loading rate of 3.5mgTSS/m^2/d, are appropriate to achieve ammonia effluent concentrations of 0.5mgNH_4-N/L, illustrating the potential to deliver anticipated future consents. Hydraulic loadings ≥0.2m/d, with associated solid loads of ≥5gTSS/m^2/d, were subjected to hydraulic overloads and subsequently limited oxygen availability within the media. The operation during the start-up period of the pilot plants is

likely to have influenced operational and performance potential, therefore findings from the study offer a conservative opinion on hydraulic limits and suggest such that once stabilised, VFWs are likely to provide passive tertiary nitrification at a reduced footprint compared to horizontal flow wetlands.

4.1 Introduction

Constructed wetlands are a low-cost, low-energy and low-maintenance alternative to conventional wastewater treatment types, and as such are becoming increasingly popular within the water industry whereby environmental and economic drivers are at the forefront of concern. Within Europe it is anticipated that stringent nutrient consents will be applied in the coming years to small wastewater treatment sites (sub 2000 person equivalent) discharging to small, ecologically vulnerable watercourses or to sites of special scientific interest, in accordance with the water framework directive (European Union, Water Framework Directive 2000/60/EC) (Griffin and Pamplin, 1998; Johnson et al., 2007). For the UK that will mean many wastewater treatment sites will require upgrading (i.e. switching to activated sludge process) or additional treatment processes (i.e. Nitrifying Submerged Aeration Filter (NSAF)) to adhere to the new standards related to ammonia and phosphorus. Both options are relatively energy intensive and so lose the near passive benefits of current technologies used on small sites such as trickling filters and/or rotating biological contactors. It is posited that vertical flow wetlands offer a sensible alternative approach to enable effective upgrading whilst retaining focus on near passive technologies (Besancon et al, 2017).

Vertical flow wetlands operate on an intermittent dosing regime, which promotes oxygen transfer within the bed through convection during the batch loading, and by diffusion of oxygen into the biofilm (Molle et al., 2006). The main limiting factor with VFW is associated with media clogging which principally occurs within the top 10cm of the main treatment media once the hydraulic, organic and solids loading rates become excessive (Knowles et al., 2011; Langergraber et al., 2003). Clogging results in water ponding on the VFW surface which is unable to fully drain between batch loads, thus restricting oxygen transfer to within the bed and preventing nitrification and organic mineralisation from taking place (Cooper, 2005). Operational practices come from the use of VFWs for treatment of secondary wastewater with recommended organic loading rates of ≤20gCOD/m^2/d and solids loading rates of 5gTSS/m^2/d for single stage systems (Winter and Goetz, 2003). In comparison, analysis of established two stage VFWs in France, which operate with multiple beds in parallel to enable resting cycles, operate with loadings up to 180gTSS/m^2/d on the first stage and up to 60gTSS/m^2/d on the second stage (Troesch et al., 2014). In addition, pilot scale trials on high rate systems have operated at loading rates up to 250gTSS/m^2/d (Millot et al., 2016) demonstrating the system can handle solid laden waters.

The hydraulic loading rate (HLR) is a large causative contributor to these factors. High HLRs often result in high batch feeding volumes, which favours oxygen diffusion into the VFW, but due to the greater water velocity through the bed, reduces the in bed detention time and the contact time between the water and biofilm (Torrens et al., 2009). Lower HLRs have the opposite effect, increasing the detention and water to biofilm contact time, and as such are favoured in secondary nitrifying VFWs where oxygen limit is the controlling factor (Molle et al., 2006). Typical HLRs for single stage VFWs treating secondary wastewater are around 0.05m/d (Brix and Arias, 2005) with much higher loading used in two stage VFWs of up to 0.37m/d as dry weather flow as they then encompass a resting rotation to remediate clogging impact through a third on, two thirds off rotation schedule (Morvannou et al., 2015). Once the beds are established with the inclusion of a 10cm sludge layer, recommended hydraulic loading rate limits are 0.9m/d during rain events and 1.8m/d for no more than once a month (Troesch et al., 2014). In such cases the bed can rehabilitate themselves during the natural rotation cycles.

In the case of tertiary applications, the contaminant load is relatively low and the water is likely to contain dissolved oxygen. Accordingly, it is proposed that standard practice from upstream operation may not fully reflect best practice when using VFW for tertiary applications. To date there is a paucity of information concerning optimal operation of VFW under tertiary application, however previous studies have used a HLR of up to 0.27m/d on each VFW operating in parallel, and an average HLR of 0.45m/d across four VFW operating in rotation (Cooper et al., 1997; Schönerklee et al., 1997). However, a recent case of a single VFW for tertiary operation incorporated a HLR of 0.006-0.08m/d with recirculation (Weedon, 2017). The current study hypothesised that to reach the full treatment capacity of a VFW under tertiary operation, a greater HLR would be beneficial as it would increase ammonia loading, although a long term balance between the optimal hydraulic loading, treatment performance and clogging control would need to be determined for successful operation. Therefore, the current study aims to determine the impact of hydraulic loading rate, ranging between 0.05m/d and 1.04m/d, on the treatment performance and degree of clogging in pilot scale VFWs. From this, the four best performing VFWs were operated in duplicate to determine the optimal hydraulic loading rate for tertiary VFW application

4.2 Materials and Methods

4.2.1 Site Description

The research was conducted on eight pilot scale VFWs on a sewage treatment works in the Midlands, UK, with a treatment capacity of 65,000 population equivalents. The site operated as a tertiary treatment works to achieve 95 percentile discharge consents of 10mgO_2/L of biochemical oxygen demand (BOD), 25mgTSS/L of suspended solids, 3mgNH_4-N/L of ammonium-nitrogen (NH_4-N) and 2mgTP/L of total phosphorus (TP). Onsite treatment processes included settlement tanks for primary treatment, trickling filters followed by humus tanks for secondary treatment with tertiary treatment being performed by both sand filters (SF) and nitrifying submerged aerated filters (NSAF), each receiving 50% of the flow. To achieve the low phosphorus consents, ferric sulphate and lime were dosed prior to, and following secondary treatment. The pilot VFWs were installed and commissioned onsite in July 2013 and were positioned to receive the humus tank effluent. To ensure delivery of fresh influent, the humus effluent was continuously pumped to a header feeding tank and surplus water returned via an overflow pipe, which discharged downstream of the influent collection.

4.2.2 Pilot Plant Design

Each pilot VFW measured 2.5m x 2.5m and was constructed above ground using glass reinforced plastic, totalling a surface area of 6.25m^2 per bed. Each wetland had a depth of 1m including (from top to bottom): 0.25m of freeboard, 0.5m of <4mm diameter concreting sand, 0.1m of 4-8mm diameter pea gravel, and 0.15m of 16-32mm diameter large gravel for drainage (Figure 4-1A). Influent was delivered to each VFW in pumped batches, via individual 5cm diameter hoses connected to a PVC pipe distribution system on the surface of each bed. The size of the distribution system, and configuration of individual feeding points, varied for each bed depending on the hydraulic load received (Table 4-1). During the initial design of the VFW, larger diameter inlet pipes with a greater number of inlets were required to accommodate the higher HLRs. During the second experimental phase, the number of inlets of VFW 1-6, with the larger diameter inlet pipework, were reduced to 8 to accommodate the lower HLRs. Roofing tiles were placed below each feeding point for even flow distribution over the surface to minimise preferential pathways. During construction 10cm diameter internal water sampling pipes, perforated to a height of 10cm and capped for water collection, were installed within each media layer of all the VFWs. The drainage system comprised two 10cm diameter perforated PVC pipes positioned on the floor of each wetland, and included passive aeration pipes extending to 15cm above the wetland surface, also acting as a system overflow (Figure 4-2B). Each VFW had a main external drainage isolation valve and a smaller side valve allowing collection of effluent samples. Wetland effluent was directed back to the humus effluent tank, downstream from influent collection, where it was directed to additional tertiary treatment prior to discharge from site. Each of the VFWs were intermittently loaded with three minute batch feeds every two hour interval, resulting in 12 feeds per wetland per day. As the VFWs were constructed above ground it was necessary to pump the feed doses to each wetland, which were controlled using individually programmed timers. During the study the VFWs remained unplanted.

The study was conducted between July 2013 and March 2014, consisting of two phases aimed at determining the effect of hydraulic loading rate on VFW performance and hydraulic behaviour. The first phase was carried out between July and September 2013 for a total of 63 days, to determine the upper hydraulic load limits, and tested a total of 13 different hydraulic loads on the eight VFWs. The second phase took place between October 2013 and March 2014, a total of 155 days, whereby the four optimum hydraulic loading rates were operated across duplicated beds.

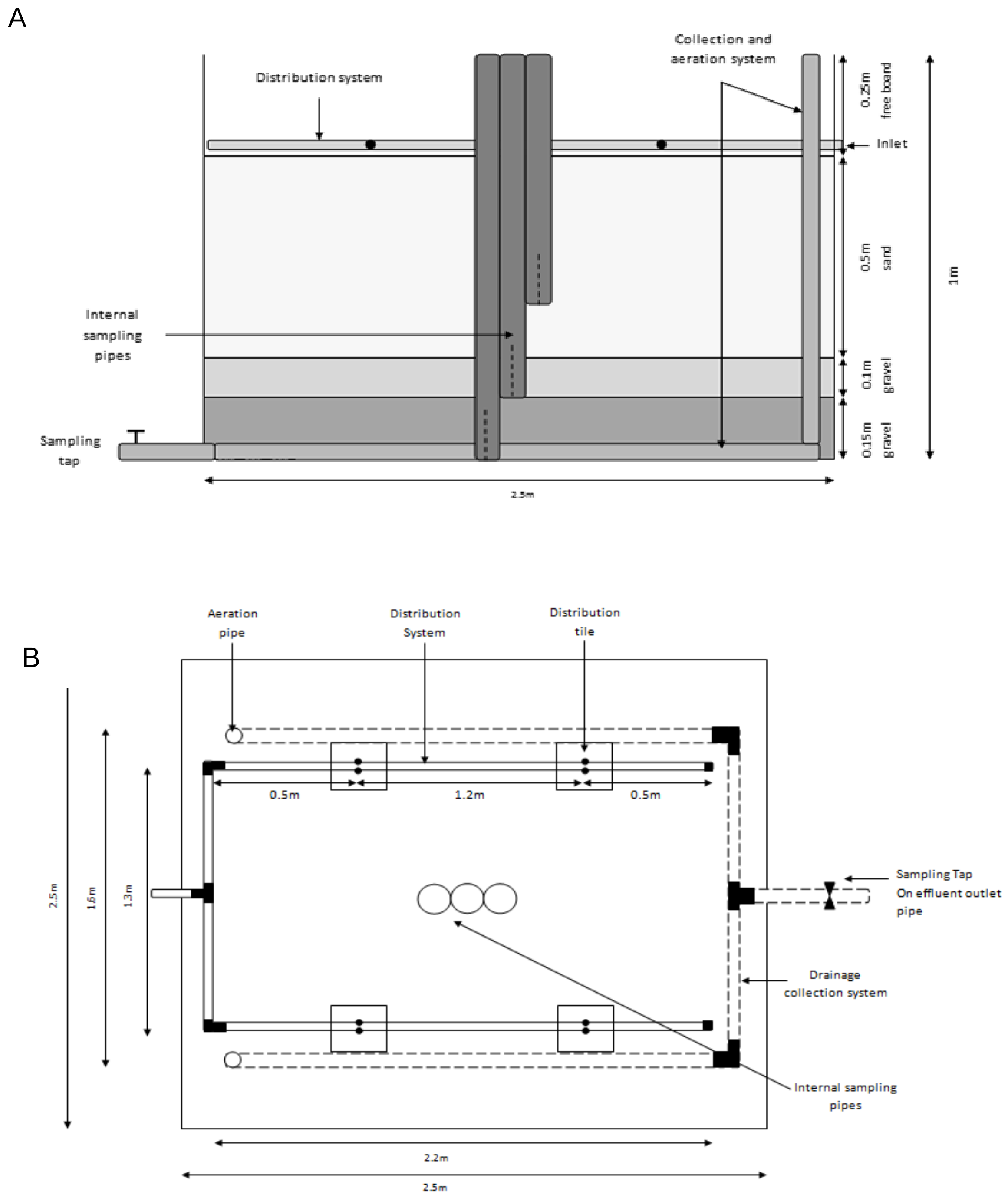


Figure 4-1 (A) Schematic view of the pilot VFW depicting media distribution and internal sampling pipes. (B) Areal view of the pilot VFW, depicting the distribution and drainage system.

Table 4-1 Design details and hydraulic loading rates applied for each experimental phase.

	Phase 1					Phase 2		
Bed ID	**HLR (m/d) (no. of samples)**		**Pipe ø (cm)**	**No. of inlets**		**HLR (m/d) (no. of samples)**	**Pipe ø (cm)**	**No. of inlets**
Bed 1	1.04 (3)	0.15 (11)	6.4	28		A) 0.15 (30)	6.4	8
Bed 2	0.92 (3)	0.05 (11)	5.1	20		A) 0.05 (30)	5.1	8
Bed 3	0.81 (9)	0.26 (5)	5.1	16		B) 0.15 (30)	5.1	8
Bed 4	0.60 (9)	0.40 (5)	5.1	10		B) 0.05 (30)	5.1	8
Bed 5	0.46 (9)	0.36 (5)	3.8	10		B) 0.19 (30)	3.8	8
Bed 6	0.32 (14		3.8	10		B) 0.07 (30)	3.8	8
Bed 7	0.19 (14)		2.5	8		A) 0.19 (30)	2.5	8
Bed 8	0.07 (14)		2.5	8		A) 0.07 (30)	2.5	8

4.2.3 Water Sampling and Analysis

Influent and effluent VFW samples were taken on a weekly basis, with two additional sample sets taken once a month, totalling 15 sampling campaigns during the first phase and 30 during the second phase. Grab samples were collected from the wetland inlet during a feed and from each outlet approximately 10 minutes after the feed commenced. Samples were collected in 1 litre plastic bottles and were transported immediately to the laboratory for same day analysis. Internal water samples were measured *in situ* via the internal sampling pipes, for pH, dissolved oxygen (DO) and redox potential (ORP) using a portable multi-meter and rugged outdoor probes (HQ40d multi-meter with pH gel electrode, ORP electrode and LDO sensor; Hach, Germany).

Influent and effluent samples were analysed for ammonium-nitrogen (NH_4-N), nitrate-nitrogen (NO_3-N), nitrite-nitrogen (NO_2-N), total phosphorus (TP) and chemical oxygen demand (COD) concentrations using colourimetric test kits (Hach, Germany). Soluble COD analysis samples were filtered through 0.45µm pore-sized filters. Alkalinity was determined through a drop count titration method (Hach, Germany). Samples analysed for 5-day carbonaceous biochemical oxygen demand ($cBOD_5$) were prepared using pre-packaged BOD nutrient buffer pillows and nitrification inhibitor, as per the manufacturer's instructions (Hach, Germany). Total and volatile suspended solids (TSS and VSS respectively), were determined using Standard Methods (APHA, 2005), using glass microfiber filter papers with a particle retention size of 1.2µm. On arrival at the laboratory, 30ml of each water sample was filtered through a 0.45µm pore size membrane and preserved with 3ml of concentrated nitric acid before being stored at 4°C for metals analysis, which was conducted every two months. These samples were analysed for nickel (Ni), lead (Pb), copper (Cu), chromium (Cr), cadmium (Cd) and zinc (Zn) using inductively coupled plasma-mass spectrometry (ICP-MS), and Iron (Fe) using flame atomic absorption spectroscopy (FAAS).

Membrane fractionation, used to differentiate between suspended, colloidal and dissolved total phosphorus and heavy metals in the influent and effluent samples, was conducted on a monthly basis. The fractionations were conducted using an Amicon 8400 stirred pressure cell (Millipore, UK), using a pressure of 3.7atm, controlled using a nitrogen gas supply. Prior to and between uses, polyethersulfone membranes of pore size 100kDa and 1kDa (Omega™, PALL life Sciences, UK), were prepared and cleaned using a wash sequence of ultrapure water, 0.1M sodium hydroxide (NaOH) and 0.1M hydrochloric acid (HCl).

4.2.4 Clogging Analysis

Wetland media percolation tests were carried out once a month to gauge the level and rate of in-bed clogging, using a modified method of the sand test described in Griggs and Grant (2000). Sand samples from each wetland were collected and placed individually in 11cm diameter plastic columns to a depth of 20cm and positioned on top of a free draining basin, using a water permeable material at the base of the column to prevent media washout. The sand samples were taken from the same location across all the VFWs and these locations were changed each month to minimise variation between sand samples whilst ensuring undisturbed sand was used for a true gauge of clogging. Each test was initiated with a one litre tap-water flush to pre-wet the samples. Percolation rates were determined by averaging the time taken for three separate loadings of 500ml tap-water to completely drain through the media sample. Sand percolation tests were conducted on clean media prior to pilot plant commission, with an average percolation time of 70 seconds (Griggs and Grant (2000) recommend between 45 and 75 seconds for the percolation of clean water through new media).

4.2.5 Statistical Analysis

Statistical analyses were conducted using Minitab V.17 software package (Minitab Inc, Pennsylvania State University, USA). One way ANOVA (p), with a significance level of 0.05, was used to determine the significant differences of solids, organics and nutrient removal efficiencies between the wetlands. Pearson product-moment correlation coefficients (r) were used to determine relationships between mass loadings and mass removals. For regression analysis, fitted line plots were used to identify the coefficients of determination (R^2).

4.3 Results and Discussion

Influent pollutant concentrations varied slightly in accordance with the season, but remained comparable over the two experimental phases (Table 4-2).

Table 4-2 Concentration range (and average) of wetland influent across the two experimental phases.

	Phase 1	Phase 2
Wastewater Parameters	Values (mg/L)	Values (mg/L)
TSS	10-33 (18.5)	11-59 (23.9)
VSS	7-21 (12.6)	8-32 (14.5)
$cBOD_5$	-	1.6-9.2 (4.7)
COD	29.0-69.2 (50.5)	32.0-76.4 (52.8)
NH_4-N	0.50-8.35 (2.5)	0.23-7.88 (2.7)
NO_2-N	0.14-0.34 (0.25)	0.13-0.85 (0.31)
NO_3-N	17.1-38.0 (32.1)	11.0-36.8 (19.6)
TP	0.56-1.55 (1.04)	0.35-2.43 (1.03)
Alkalinity	25-60 (38.3)	15-120 (70)
DO	5.8-6.9 (6.3)	5.5-7.3 (6.6)

4.3.1 Effect of Loading Rate on Hydraulic Acceptance

The upper hydraulic limits of the unplanted VFWs were tested during the first phase by direct comparison with beds operated at HLRs of between 0.05m/d and 1.04m/d. The VFWs were defined as having reached their hydraulic limit once the surface water could no longer naturally drain between feeds and the water levels reached the system overflow, 15cm above the media. Once this condition was met, the wetland surface was perforated to relieve clogging and left to drain freely with the soiled surface media (1-2cm) removed and the HLR reduced. The functional duration before the hydraulic load limit was achieved, increased with decreasing HLRs. For instance, operational durations of up to 13 days were observed for HLRs of 1.04m/d and 0.92m/d, up to 45 days for HLRs of 0.81m/d, 0.60m/d and 0.46m/d and 63 days for a HLR of 0.32m/d. These equate to solids loading rates, based on the mean solids concentration, of 19.2gTSS/m^2/d and 17gTSS/m^2/d, respectively, for the 13 days operating duration; 15gTSS/m^2/d, 11.1gTSS/m^2/d and 8.5gTSS/m^2/d, respectively, for the 45 day operating period of and 5.92gTSS/m^2/d for the 63 day operating period. These reported levels exceed the recommended solids loading rates for use of VFWs for upstream applications where 5gTSS/m^2/d is considered the upper loading limit to avoid clogging in single pass systems (Winter and Goetz, 2003). The maximum operational HLR at which no clogging was observed was 0.19m/d, which was equivalent to a solids loading rate of 3.5g/m^2/d. Overall, the results are congruent with accepted practice and suggests that tertiary applications follow similar lines even though the contamination load is significantly different. However, the current data reflects early operation of unplanted VFWs and so can be considered a worst case. For instance, previous investigations into the use of two stage VFWs for treating raw sewage have indicated that the systems require up to 2 years to acclimatise and that hydraulic limits are likely to be reached until the system stabilises (Gomez, 2016). The system stabilises once the microbial biomass becomes fully active as it aerobically degrades the organic matter, rehabilitating the systems between feeds such that a steady state can be established. The impact of stabilisation can be observed through the reported higher solids loading rates managed with mature tertiary wetlands where up to 10.3g/m^2/d has been possible (Weedon, 2003) equating to a HLR in the current case of 0.56m/d. Additionally, emergent wetland plants can offer supplementary treatment through the direct uptake of nutrients, such as ammonium, nitrogen and phosphorus, and heavy metals, with increased performance efficiency of 10% for ammonium-nitrogen removal, 30% for nitrogen removal and 5% for phosphorus removal reported for planted systems, when compared to unplanted systems (Lana et al., 2013; Abou-Elela and Hellal, 2012; Macci et al., 2014; Zhang, Rengel and Meney, 2007).

The second phase of the experiments were conducted on reduced HLRs of between 0.05m/d and 0.19m/d, the lower end reflecting current practice with single stage VFWs for secondary wastewater treatment (Brix and Arias, 2005). Effective operation occurred across all the tested HLRs with clogging only observed in two beds: A)0.15m/d and B)0.19m/d which reached their hydraulic limit on three separate occasions during the 155 days of operation, believed to be influenced by previous clogging and heavy rainfall. Percolation tests after 6 months of operation in the second experimental phase showed all eight of the VFWs had percolation times <460 seconds, ranging between 231 and 453 seconds. This compares to percolation times of 756 and 2174 seconds when using the higher HLRs of 0.81m/d, and 0.46m/d, respectively (Table 4-3). Reed and plant detritus is considered to contribute to wetland surface clogging therefore special effort was made to ensure wetland surfaces were free of any plant type, and as a result approximately $0.06m^3/m^2$ of plant material was removed from wetland surfaces over the course of the study.

The media used in the current study was based on standard UK practice for single stage VFW for secondary treatment (Hudson, 2011) and is comparable to literature recommendations (ø0-4 mm) (Brix and Arias, 2005). In previous studies with similar sized media (ø0.5-9 mm) clogging occurred at HLRs of 0.24m/d (Weedon, 2003) at full scale and at lower HLRs of 0.1m/d and 0.15m/d at pilot scale when media of 1-4mm and 0.06-4mm were used (Langergraber et al., 2003). Larger media could be employed to reduce clogging risk especially in the upper layer but must be balanced against treatment performance and would require sufficient contact between the contaminants and the biofilm. Higher stable HLRs require active biofilms to maintain the condition of the bed, which requires sufficient time during which the bed is drained. For instance, recommendations for two stage VFW treating raw sewage is that the beds must be drained for at least 14.5 hours/day. Further, when operating at high solids loading rates it is recommended to rotate operation between parallel beds to ensure sufficient conditioning time between feed cycles (Torrens et al., 2009).

Table 4-3 Average and range of percolation times through wetland media during the first and second experimental phase

Bed ID	HLR Phase 1		26/07/13	19/08/13	30/08/13		HLR Phase 2	19/10/13	18/11/13	13/01/14	18/02/14	10/03/14
Bed 1	1.04m/d	**Average**	**456**	Flow changed to 0.15m/d								
		Range	418-516									
Bed 1	0.15m/d	**Average**		**388**	**407**		A)0.15m/d	**2415**	**155**	**774**	**744**	**429**
		Range		274-460	323-478			2198-2625	126-179	706-896	170-1203	202-656
Bed 2	0.92m/d	**Average**	**285**	Flow changed to 0.05m/d								
		Range	243-303									
Bed 2	0.05m/d	**Average**		**390**	**388**		A)0.05m/d	**675**	**218**	**355**	**406**	**235**
		Range		332-521	304-447			497-865	178-281	310-385	200-692	120-410
Bed 3	0.81m/d	**Average**	**334**	**462**	**756**		B)0.15m/d	**242**	**238**	**266**	**272**	**231**
		Range	323-348	423-506	472-908			204-286	182-280	240-298	218-363	174-336
Bed 4	0.60m/d	**Average**	**357**	**306**	**289**		B)0.05m/d	**226**	**572**	**586**	**548**	**321**
		Range	307-416	269-349	203-368			165-274	435-706	497-645	274-767	156-777
Bed 5	0.46m/d	**Average**	**344**	**430**	**2174**		B)0.19m/d	**191**	**199**	**971**	**753**	**424**
		Range	287-394	323-508	1985-2314			183-213	140-240	694-1318	535-997	216-658
Bed 6	0.32m/d	**Average**	**351**	**120**	**386**		B)0.07m/d	**262**	**308**	**506**	**778**	**237**
		Range	253-406	65-145	310-454			200-299	240-358	359-658	534-1065	117-453
Bed 7	0.19m/d	**Average**	**205**	**319**	**339**		A)0.19m/d	**145**	**181**	**1188**	**628**	**453**
		Range	193-215	294-346	272-392			125-162	147-228	1047-1265	299-737	204-829
Bed 8	0.07m/d	**Average**	**141**	**231**	**244**		A)0.07m/d	**250**	**259**	**263**	**402**	**445**
		Range	125-158	122-314	216-291			170-302	206-307	259-265	274-664	240-685

4.3.2 Effect of Hydraulic Loading on Nitrification Capacity

Ammonium-nitrogen removal occurred within 8 days of operation, achieving removal efficiencies of between 96% and 99% for VFWs operating with HLRs of 0.07m/d, 0.19m/d, 0.32m/d, 0.46m/d, 0.60m/d and 0.81m/d. In comparison, the higher HLRs resulted in reduced removal efficacies of 36% and 55% for the 0.92m/d and 1.04m/d trials respectively. Nitrate-nitrogen concentrations remained consistent between influent and effluent within the first 8 days, suggesting ammonium-nitrogen was not removed by nitrification, but by another mechanism such as adsorption onto the fresh media, of which sand has an ammonia absorption capacity of 4.9mgNH_4-N/L per gram of sand (Azhar and Aimi Shaza 2012).

The first experimental phase saw the greatest ammonium-nitrogen removal within the four lightest hydraulically loaded beds, producing mean effluent concentrations of 0.02mgNH_4-N/L, 0.11mgNH_4-N/L, 0.14mgNH_4-N/L and 0.02mgNH_4-N/L for HLRs of 0.05m/d, 0.07m/d, 0.15m/d and 0.19m/d, respectively (Figure 4-2). Nitrate-nitrogen effluent concentrations achieved a maximum increase of 5% during phase 1, suggesting that nitrification was occurring. However the nitrifying biofilm may not have been fully established (Paing et al., 2015; Weedon, 2003). The greatest ammonium-nitrogen mass removal was achieved in VFWs with HLRs of 0.6m/d and 0.8m/d, with removals of 1.06gNH_4-N/m^2/d and 1.29gNH_4-N/m^2/d, respectively. However, ammonium-nitrogen removal in VFWs receiving a HLR of 0.26m/d and above was more varied and corresponded to variation in dissolved oxygen patterns observed within the media (Figures 4-3A). The overall impact was that the target limit of 3mgNH_4-N/L was exceeded for beds receiving HLRs of 0.26m/d, 0.4m/d and 0.81m/d (Figure 4-2A).

During the second experimental phase ammonium-nitrogen removal remained high in six of the VFWs, achieving effluent concentrations as low as 0.024mg/L (99% removal efficiency). The two poorest performing 'problematic' VFWs, A)0.15m/d and B)0.19m/d, achieved ammonium-nitrogen effluent concentrations of 0.653mg/L (75%) and 0.943mg/L (72%), respectively, with both 95th percentiles exceeding the sites ammonium-nitrogen consent (Figure 4-2B). Ammonium-nitrogen mass removals increased with increasing mass loads, with the exception of the two problematic VFWs. The dissolved oxygen availability in the higher loaded beds (0.15m/d and 0.19m/d) was lower than in the lighter loaded beds, with corresponding redox potential patterns observed (Figure 4-3B, 4-4B). Regression analysis, using a fitted line plot, showed poor linear relationships between dissolved oxygen concentrations in the main treatment media and the NH_4-N load removed across all VFWs (R^2 = 7.3%, P=0). A nitrogen mass balance (of ammonium-nitrogen, nitrite-nitrogen and nitrate-nitrogen) showed nearly-complete nitrification in all but the two problematic beds, with at least 96% removal of NH_4-N, 73% removal of NO_2-N and an increase in NO_3-N. Total nitrogen (TN) (calculated as the sum of NH_4-N, NO_2-N and NO_3-N) showed a mass increase from 6.3g/d for HLR 0.05m/d to 26.8g/d for HLR 0.19m/d, with NO_3-N contributing >96% of the TN. This was expected due to the predominantly aerobic nature of VFWs. However, in addition to proposed stringent NH_4-N and TP consents on wastewater treatment works, it is anticipated TN consents will become more frequent in the future. Therefore, it may become necessary to follow VFW treatment with a predominantly anaerobic treatment, such as horizontal flow wetlands, or by use of a recirculating flow of treated effluent to the VFW inlet to reduce NO_3-N and subsequently TN concentrations (Laber et al., 2003). Despite the two problematic beds not achieving full nitrification, the nitrogen mass balance showed that some nitrification did occur. Interestingly, in all cases (across both experimental phases) the alkalinity concentrations had increased between the influent and effluent, with increases of up to 110mg/L (275%) during phase 1, and 78mg/L (112%) during phase 2.

Overall the best performing VFWs were associated with the lower HLRs which is consistent with previous studies using VFWs for upstream applications (Bancolé et al., 2003; Molle et al., 2006; Torrens et al., 2009). Sustained low effluent ammonia was achieved up to loading rates of ~0.4gNH_4-N/m^2/d and is comparable to previous reports where ≥90% removal was achieved up to a loading rate of 0.4gNH_4-N/m^2/d (Schönerklee et al., 1997). The highest hydraulically loaded VFWs received ammonium-nitrogen loading rates of ≥2gNH_4-N/m^2/d, but still performed a mass removal of up to 1.3gNH_4-N/m^2/d before becoming hydraulically overloaded. Additionally, ammonium-nitrogen mass removal is seen to be proportional to the influent NH_4-N loading across both experimental periods, indicating that ammonium-nitrogen loading was not a performance limiting factor during the study. Similar effluent concentrations have been reported for full scale aerated horizontal flow wetlands (AHFWs) (Butterworth et al, 2013) and pilot scale VFWs after activated sludge (Besancon et al, 2017). However, the upper loading rate experienced in the AHFW was much greater at 12.5gNH_4-N/m^2/d due to the higher HLRs operated at 0.46m/d (Butterworth et al., 2016). In the current case, stable low ammonia effluent concentrations were associated with stable high residual DO availability, with reduction in nitrification linked to a low internal DO within the bed resulting from surface clogging.

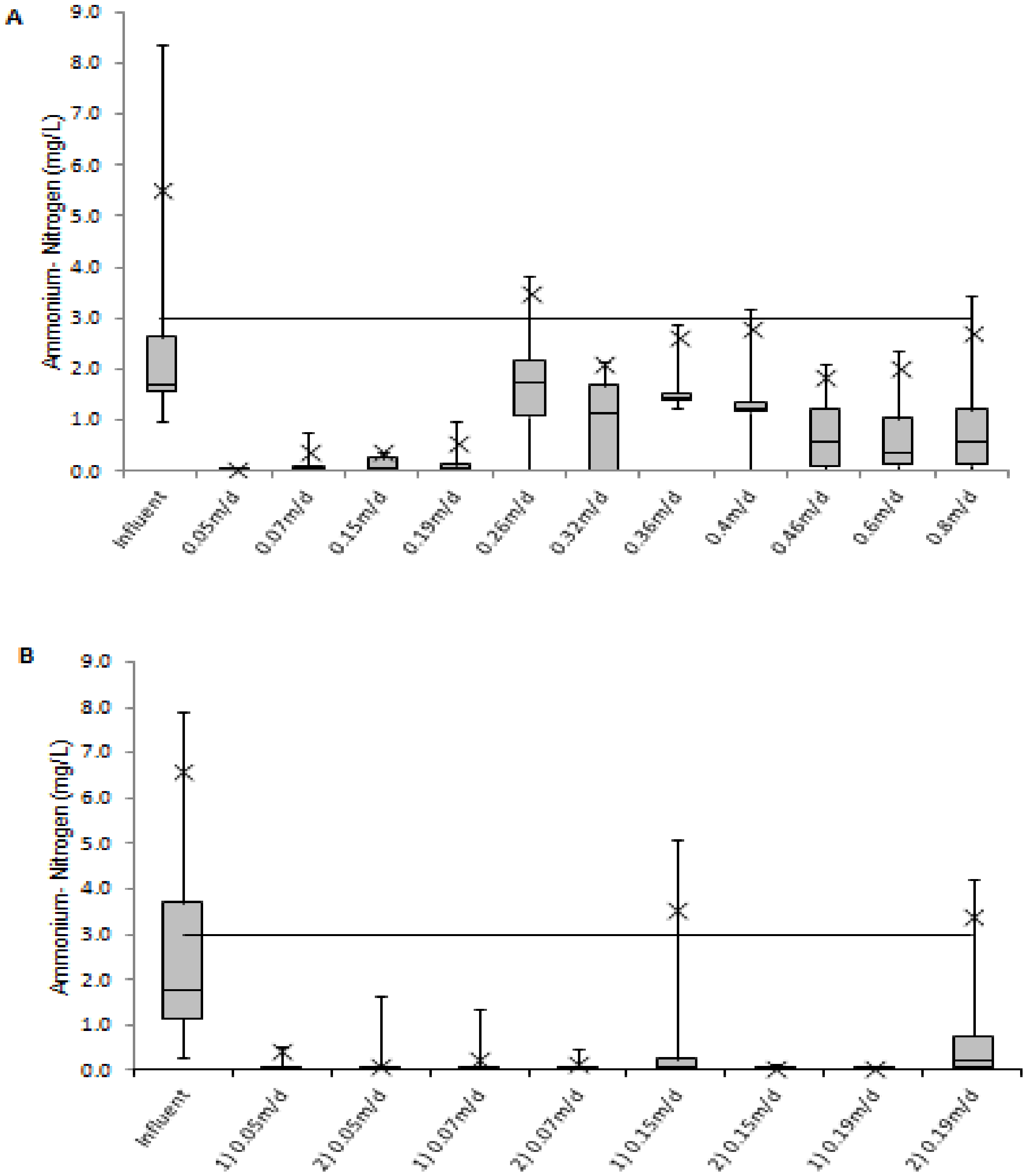


Figure 4-2 Box and whisker showing ammonium-nitrogen influent and effluent concentrations for (A) experimental phase 1 and (B) experimental phase 2. The box represents the median, upper and lower quartiles; the whiskers represent the minimum and maximum; the cross represents the 95th percentile and the line represents the site NH_4-N consent.

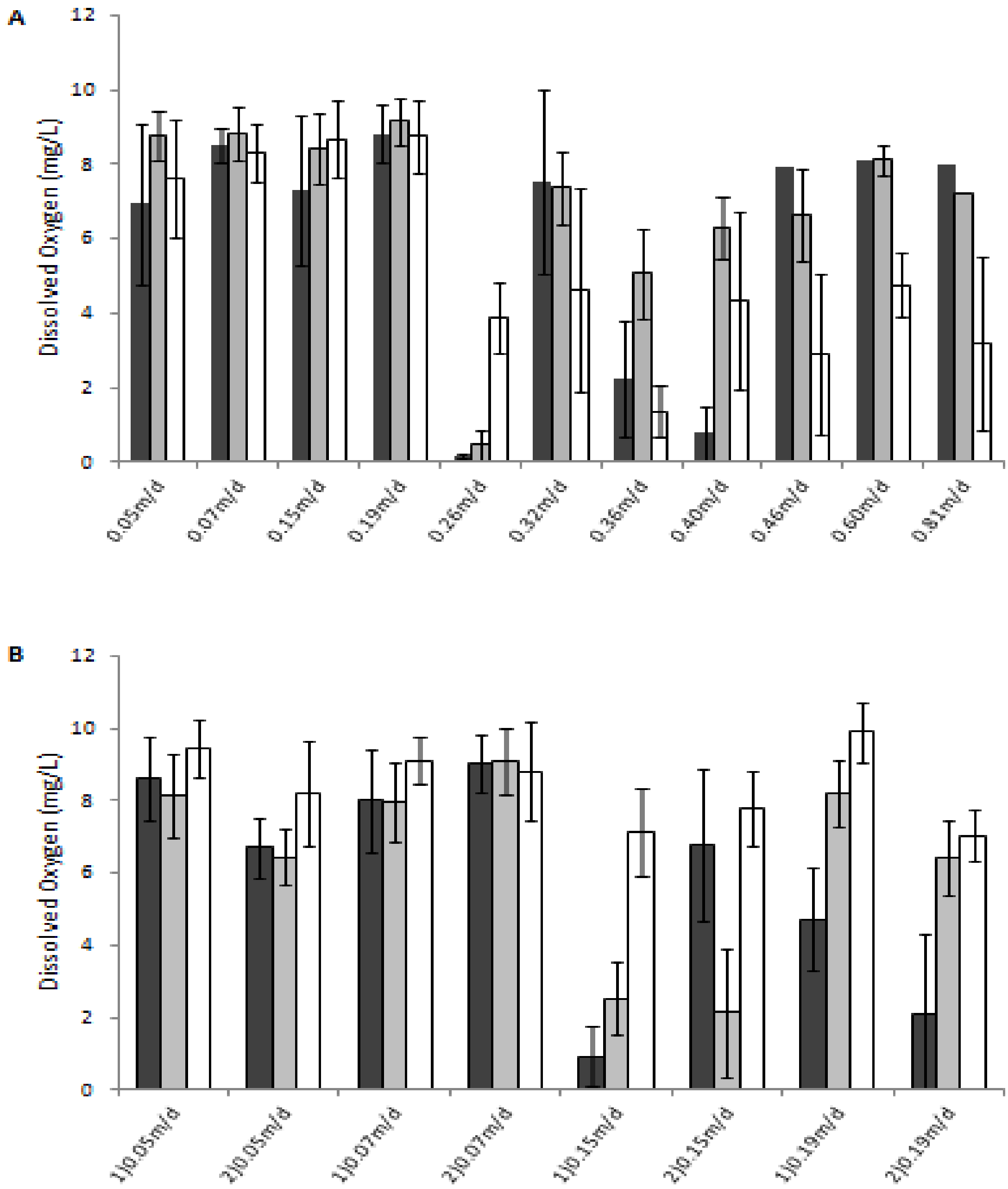


Figure 4-3 Dissolved Oxygen concentrations within the VFW internal sampling points for (A) the first experimental phase and (B) the second experimental phase. The dark grey represents the main treatment media; the light grey represents the transition media; the white represents the drainage media. Errors bars show the standard deviation.

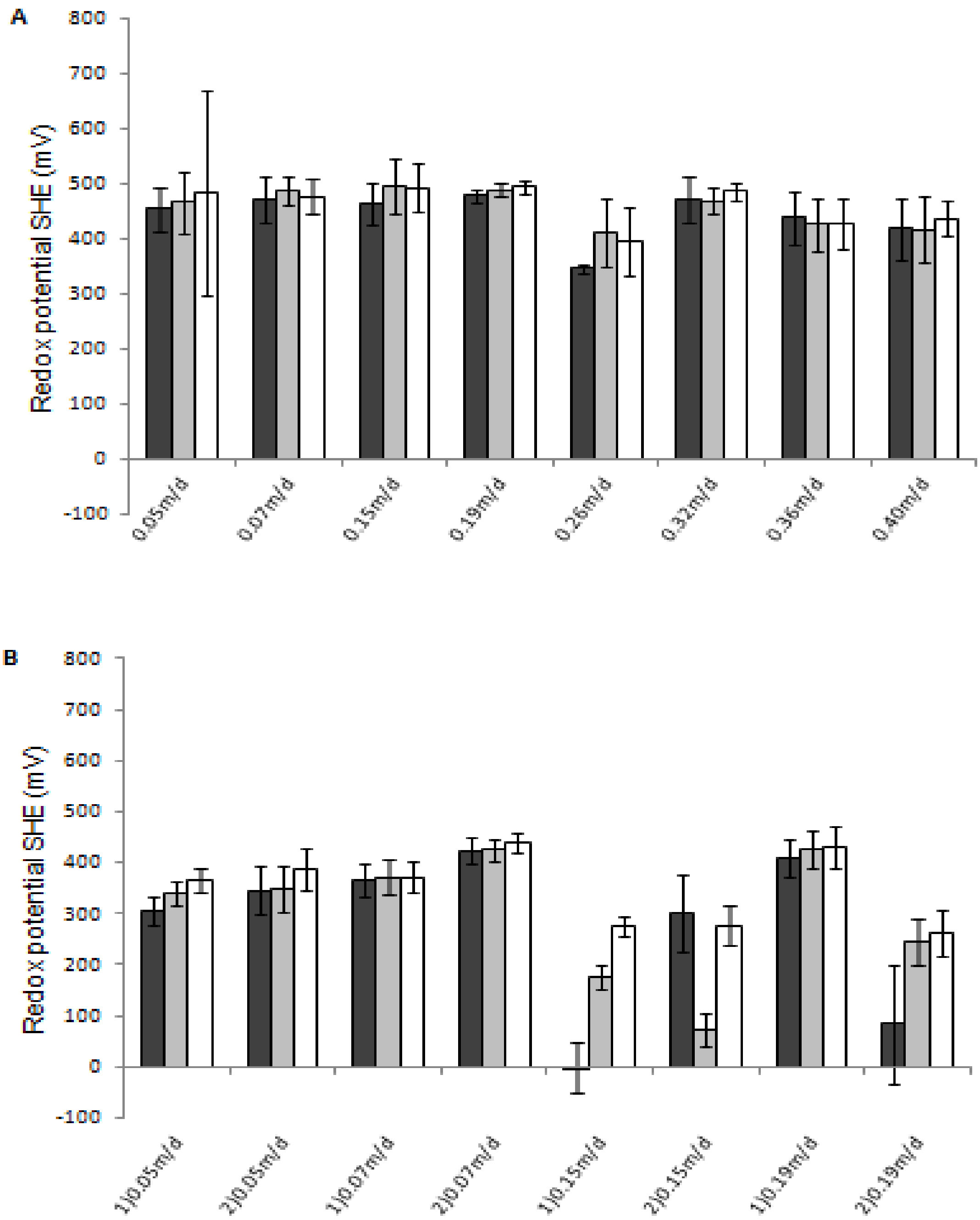


Figure 4-4 Redox potential within the VFW internal sampling points during (A) the first experimental phase and (B) the second experimental phase. The dark grey represents the main treatment media; the light grey represents the transition media; the white represents the drainage media. Errors bars show the standard deviation. HLRs of above 0.40m/d were omitted from (A) due to small sample sizes.

4.3.3 Effect of Hydraulic Loading Rate on Solids and Organic Pollutant Removal

Total suspended solids removal occurred across all VFWs, with the greatest efficiency demonstrated in the four lightest hydraulically loaded beds (0.05-0.19m/d) in the first experimental phase, resulting in average effluent suspended solids concentrations of between 2.78 and 3.71mgTSS/L equating to removal efficiencies of 80-86% (Figure 4-5A). Effluent suspended solids became more varied once the HLR was ≥0.26m/d reflecting solids release prior to or post hydraulic overloading. Total suspended solids mass removal showed a strong linear relationship to solids loading for loads up to 6gTSS/m^2/d (ranging between r=0.872, p=0 and r=0.988, p=0), equating to a HLR of up to 0.32m/d. For solids loading above 6g/m^2/d, mass removal became much more varied and unpredictable, often achieving less than 50% removal. Effluent TSS concentrations during the second experimental phase had a greater variability within the four VFWs receiving the two highest HLRs (Figure 4-5B), with average concentration removal of between 13.3 and 20.9mgTSS/L, compared to concentration removal of between 20.5 and 21.6mgTSS/L for the VFW's lightest loaded VFWs. Statistical analysis (one-way ANOVA, with significant differences determined as $p<0.05$) was conducted between each pair of duplicated VFWs showing TSS concentration removals were not statistically different within the three lightest loaded VFWs (0.05m/d, p=0.438; 0.07m/d, p=0.738; 0.15m/d, p=0.075) but were significantly different between the VFWs receiving a HLR of 0.19m/d (p=0.022). Strong correlations were observed between the TSS mass removal and TSS loading (with the greatest as r= 0.994, p=0 and the weakest as r=0.533, p=0.003) during the second experimental phase, which is in agreement with TSS correlation results from the first phase.

During the first experimental phase, the chemical oxygen demand concentration removal was greater within the lightest loaded VFWs, achieving mean removal of between 30.6mgCOD/L and 33.1mgCOD/L, corresponding to removal efficiencies of between 60-66%. The VFWs receiving HLRs ≥0.26m/d produced greater variance in COD effluent concentrations with significantly lower removal efficiencies (Figure 4-6A). Chemical oxygen demand mass removal had a linear relationship to COD loading rates (r=0.797, p=0) for the entire range of HLRs tested, indicating the VFW potential to receive greater COD loads. During the second experimental phase, COD concentration removal was similar across the four different HLRs, with the exception of A)0.15m/d and B)0.19m/d, both achieving concentration removals of 28mgCOD/L, with the remaining VFWs achieving removal between 35.7 and 40.4mgCOD/L (Figure 4-6B). One-way ANOVA showed statistical similarities, in COD load removal between the VFWs with paired HLRs during the second experimental phase, was observed in the VFWs with HLRs of 0.05m/d, 0.07m/d and 0.15m/d (0.05m/d, p=0.438; 0.07m/d, p=0.738; and 0.15m/d, p=0.075). Analysis for soluble COD was conducted three times during phase 2, showing an average concentration of 39.6mgsCOD/L within the influent (65% of COD being soluble), and effluent concentrations ranging between 7.7 and 14.2sCODmg/L (between 56 and 86%).

Five samples were analysed for $cBOD_5$ during study, all of which were during the second phase. Due to low influent $cBOD_5$ concentrations, dissolved oxygen within the diluted samples did not decrease by 2mg/L over a 5 day incubation period. Undiluted effluent samples contained $cBOD_5$ concentrations of between 1.3 and 1.9mg/L (72% and 60% efficiency, respectively), with no statistically significant difference between the VFWs with paired HLRs during the second experimental phase (p>0.05). The results build on previous studies that have shown coupling VFW post activated sludge to be an effective polishing process generating mean CODs of less than 30mgCOD/L and suspended solids of less than 10mgTSS/L (Besancon et al, 2017). In fact, comparison with a membrane bio-reactor (MBR) demonstrated that the effluent quality of VFW post activated sludge was comparable in all parameter bar coliforms. A comparison of performance efficiencies between the VFWs in this study and tertiary AHFWs with the same hydraulic loading rate show the VFW were able to produce average TSS effluent concentrations of <10mgTSS/L compared to 14±9.4mgTSS/L produced by the aerated HFW. The VFW also produced lower effluent concentrations of $cBOD_5$ when compared to the tertiary AHFW (4.3±3.19mg/L). However $cBOD_5$ was only analysed during the second experimental phase of this study, and so operated at a lower hydraulic loading rate than that of the AHFW (0.27m/d). (Butterworth et al, 2013).

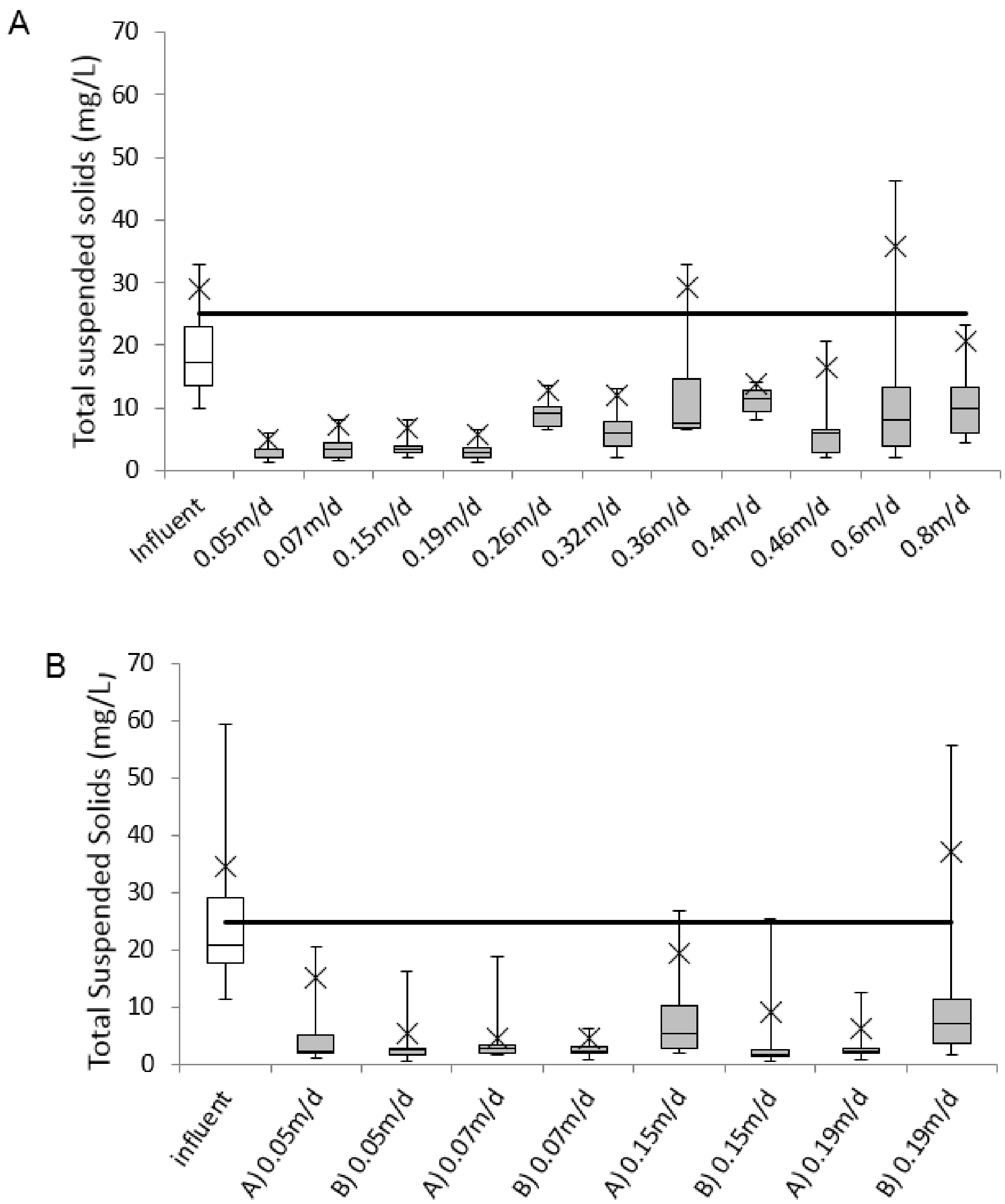
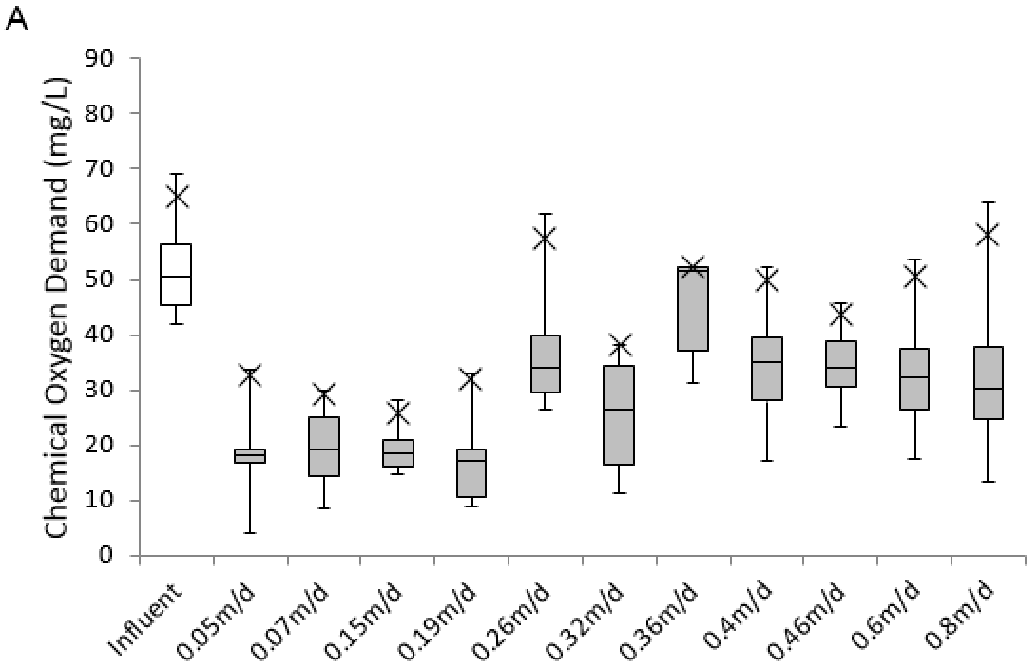


Figure 4-5 (A) TSS influent and effluent concentrations for different HLRs during phase 1. Results from HLRs of 0.91 and 1.04m/d were omitted due to small sample sizes (*n*=3). (B) TSS influent and effluent concentrations for the duplicated range of HLRs during phase 2.



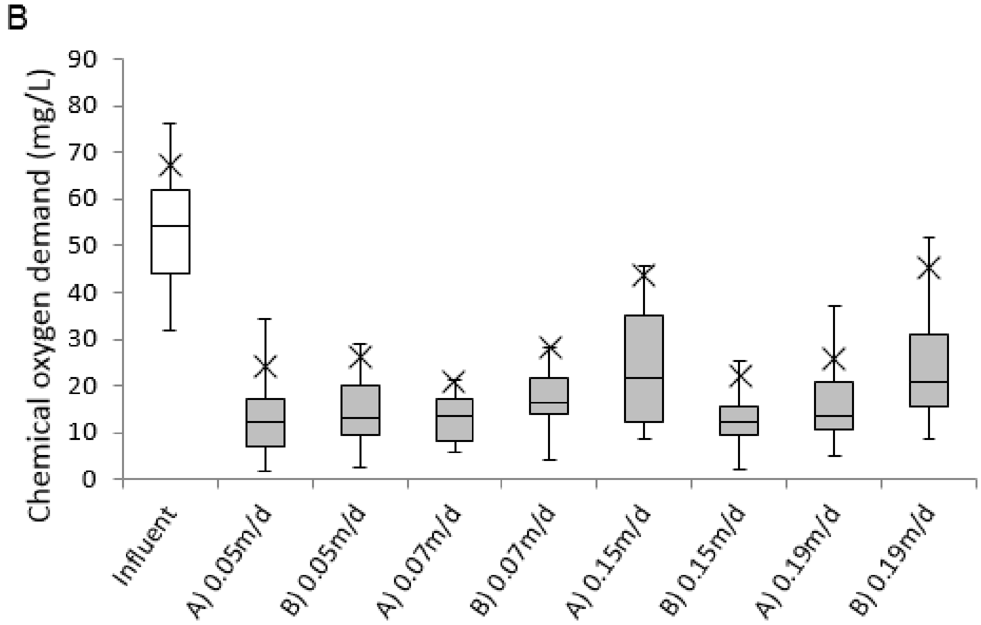


Figure 4-6 (A) COD influent and effluent concentrations for different HLRs during phase 1. Results from HLRs of 0.91 and 1.04m/d were omitted due to small sample sizes (*n*=3). (B) COD influent and effluent concentrations for the duplicated HLRs during phase 2.

4.3.4 Effect of Hydraulic Loading Rate on Phosphorus and Metal Removal

During the first experimental phase, total phosphorus removal occurred within all VFWs, with the greatest performance efficiency achieved in the four lightest hydraulically loaded VFWs (0.05m/d-0.19m/d), achieving removal efficiencies of between 78-87%. Within VFWs with HLRs of 0.6m/d and 0.8m/d TP removal was much less varied than in other VFWs (Figure 4-7A). Total phosphorus mass removal had a strong linear relationship to mass loading (R^2=94.1%, r=0.970, P=0), with the majority of mass loadings under 0.4g/m^2/d. During the second experimental phase, total phosphorus concentration removal was greatest in the four VFWs receiving the two lowest HLRs, with average removal of between 0.820mgTP/L and 0.904mgTP/L (80-88% efficiency). The VFWs receiving HLR's of 0.15m/d and 0.19m/d had more varied concentration removal, particularly in A)0.15m/d and B)0.19m/d, with average removal of 0.614mgTP/L (60%) and 0.558mgTP/L (54%), respectively (Figure 4-7B). However, one-way ANOVA statistical analysis revealed there is no significant difference between concentration removal capabilities of both VFWs receiving a HLR of 0.15m/d (p=0.095), when considering a confidence level of 0.05. Strong linear relationships (ranging between r=0.863, p=0 and r=0.998, p=0) between mass removal and mass loading were observed in seven of the eight VFWs with the exception being B)0.19m/d (r=0.322, P=0.088). Membrane fractionation of TP was conducted five times during the second experimental phase, with average influent TP composed of 81% particulate, 7% colloidal and 12% dissolved fractions. Removal of the particulate fraction contributed the greatest overall TP removal across all HLRs, with several of the VFWs achieving complete particulate removal during December, January and February (Figure 4-8).

Heavy metals analysis was only conducted during the second experimental phase (*n=17*), with average influent zinc, copper and iron concentrations of 31.8µg/L, 6.5µg/L, and 1.7µg/L respectively. Cadmium, chromium, nickel, and lead concentrations were all below the ICP-MS limit of detection (1µg/L). Excluding the two VFWs that were unstable, effective removal of the metals was observed with complete removal of iron (effluent concentration below detection limit) and 68% for zinc and 60% for Copper representing removals of 21.7µgZn/L and 3.9µgCu/L respectively. Previous trials with VFW post activated sludge showed similar levels of removal for copper but much poorer removal of zinc (Besancon et al, 2017). The latter reflect potential complexation and solubilisation with extracellular-polymeric substances (EPS) which occur at higher concentrations within the activated sludge process compared to trickling filters (Lester and Sterritt, 1985). Reduced performance was observed in the beds that suffered hydraulic problems with removals reducing to 31% and 22% for zinc and copper respectively.

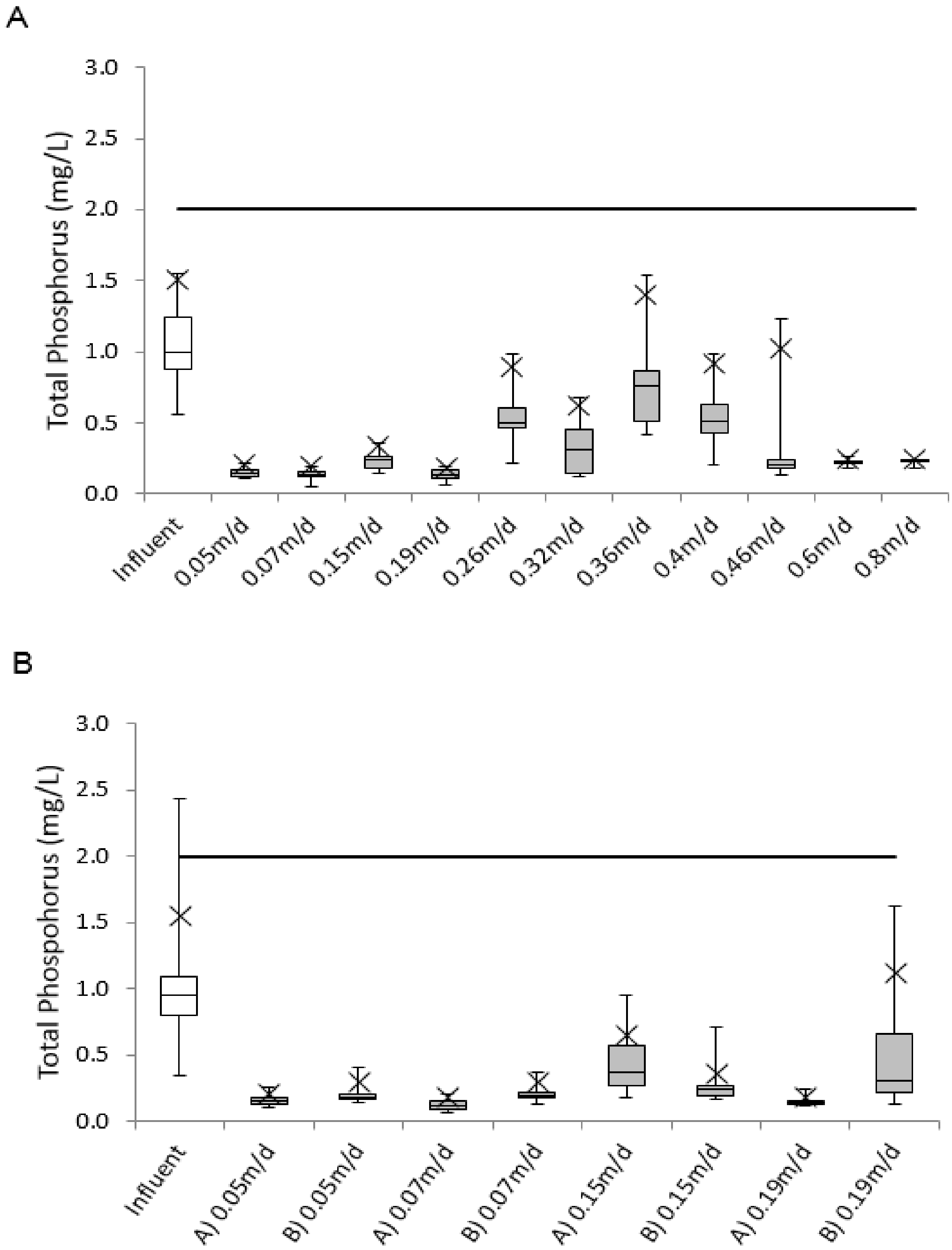


Figure 4-7 (A) TP influent and effluent concentrations for different HLRs during phase 1. Results from HLRs of 0.91 and 1.04m/d were omitted from the graph due to small sample sizes (*n*=3). (B) TP influent and effluent concentrations for the duplicated range of HLRs during phase 2. The box represents the median and the upper and lower quartiles, the whiskers represent the minimum and maximum. The x represents the 95th percentile and the solid line shows the current site consent.

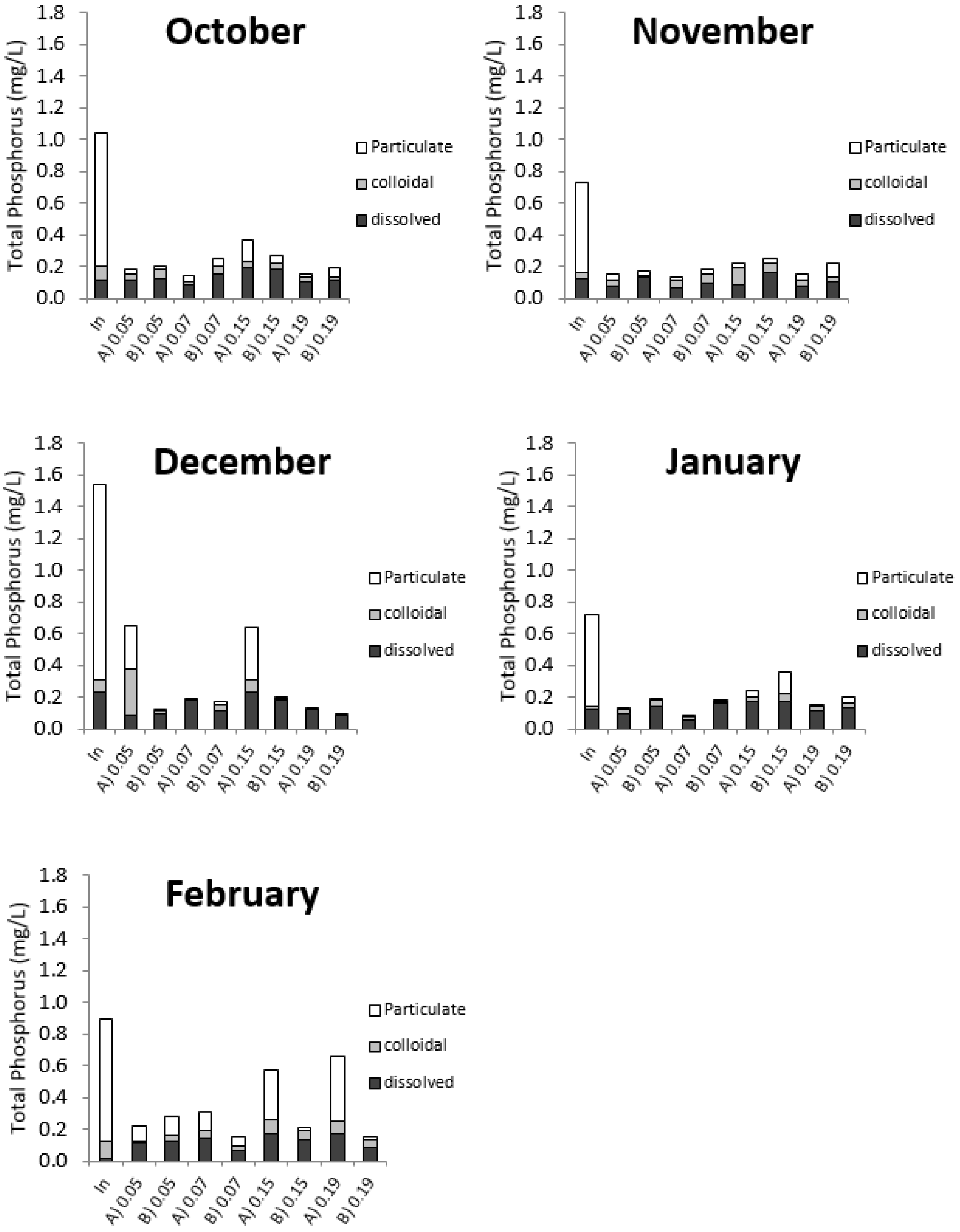


Figure 4-8 Particulate, colloidal and dissolved fractions of influent and effluent total phosphorus for months October to February.

4.4 Conclusion

Treatment performance achieved by the VFWs demonstrated the potential to deliver the anticipated future nutrient consents. In particular, VFWs under stable operation demonstrated the ability to deliver an effluent concentration of 0.5mgNH_4-N/L. The performance of the system was dependent on enabling sufficient dissolved oxygen to be maintained throughout the bed. This set a limiting hydraulic loading rate of below 0.2m/d, which equated to a solids loading rate of 3.5gTSS/m^2/d. Operation above this HLR resulted in solids loading rates exceeding 5gTSS/m^2/d and subsequent hydraulic overloading. As only one of each pair of duplicated VFWs receiving HLRs of 0.15m/d and 0.19m/d in the second experimental phase experienced hydraulic overloading, it is assumed that localised surface flooding occurred due to a combination of previous clogging and heavy rainfall.

The operating limits of the HLR are above those typically used with single stage VFW for secondary treatment but below the levels used for raw treatment when rotational cycles are included. Furthermore, the levels are at the low end of standard practice when using horizontal flow wetlands (0.2-0.4m/d). However VFWs deliver the ammonia removal whilst retaining the near passive operation of wetlands, an attribute the horizontal systems do not possess. Despite reaching hydraulic capacity, NH_4-N loading was not a performance limiting factor therefore supporting the research hypothesis. Considering the future outlook, it is important to reflect that during the current study unplanted beds within the first year of operation were used. Accordingly, the reported data can be expected to offer a conservative opinion on hydraulic limits such that the outlook suggests that once stabilised, VFWs are likely to offer passive tertiary nitrification at reduced footprint compared to HFWs.

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Chapter 5

Impact of Dosing Frequency on the Nitrification Potential within Tertiary Vertical Flow Wetlands

5 Impact of Dosing Frequency on the Nitrification Potential within Tertiary Vertical Flow Wetlands

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Abstract

Vertical flow wetlands (VFWs) are ideal nitrification treatment processes as they operate under predominantly aerobic conditions, enhanced through the use of an intermittent feeding regime to increase oxygen transfer potential. The dosing frequency of the intermittent regime has a significant impact on the treatment capacity and operation stability of a VFW and as such should be optimised to the given application. To address the knowledge gap in understanding the impact of dosing frequency on the treatment performance and clogging potential in tertiary VFW systems, a six month pilot plant trial was conducted using six different dosing frequencies ranging between 4 and 45 feeds per day. The capability of the VFWs to produce effluent concentrations of ≤1.6mgNH_4-N (0.56mgNH_4-N/L omitting outliers) of ammonium-nitrogen, ≤16.3mgTSS/L of suspended solids, ≤25.1mgCOD/L of COD and ≤0.46mgTP/L of total phosphorus was observed across all dosing frequencies. This was comparable to, if not greater than, the treatment efficiencies observed for the current onsite conventional tertiary treatments and show potential to achieve future discharge consents. The greatest overall performance in terms of hydraulic behaviours and treatment efficiency, were achieved within both the highest and lowest dosing frequencies, with no clear relationships being established, suggesting the dosing frequency is not a critical design choice when considering VFWs for tertiary application.

5.1 Introduction

The application of constructed wetlands for tertiary treatment of municipal wastewater is traditionally accomplished with horizontal flow wetlands (HFW). Horizontal flow wetlands are a passive treatment option for suspended solids, organic matter and nitrate removal. Oxygen transfer in HFW systems is poor, such that the bed generally operates with dissolved oxygen concentrations below 0.5mg/L resulting in limited nitrification (Butterworth et al., 2013). The need for tertiary nitrification is becoming more commonplace especially on small wastewater treatment works (sub 2000 PE) where sub 3mg/L ammonia concentrations are becoming required. Accordingly, HFW can be adapted to include artificial aeration in order to provide the required oxygen for nitrification. To illustrate, a recent full scale trail comparing aerated and non-aerated HF wetland reported removal efficiencies of 99% up to a maximum loading rate of 10gNH_4-N/m^2/d for the aerated bed compared to a removal efficiency of 59% up to a maximum loading rate of 1.6gNH_4^+-N/m^2/d for the unaerated bed (Butterworth et al, 2013). The technology adaptation is becoming standard with expectation of the ability to meet a 1mg/L effluent ammonia in tertiary applications and a 3mg/L when used for secondary or combined storm water/ tertiary treatment (Butterworth et al., 2016). However, the technology requires artificial aeration and hence loses its passive attributes with estimates that such systems use as much energy as activated sludge on a per PE basis (Butterworth, 2014). Further, concerns have been raised about a lack of resilience to spiked loads and solids retention if the bed operates with a water level above the media. Vertical flow systems offer an alternative that enable oxygen transfer via intermittent loading with reported oxygen transfer rates of 28-100gO_2/m^2/d (Cooper, 1999) and hence retain the passive attribute whilst enabling nitrification. In fact, VFW systems are already commonly utilised for treatment of raw or primary effluent (Kadlec and Wallace, 2009) with typical average effluent concentrations of 10±10TSSmg/L, 6±4mgBOD5mg/L and 5±6mgNH_4-N/L for TSS, BOD_5 and NH_4-N respectively based on composite samples when applied to raw wastewater (Paing et al., 2015).

The key to the efficacy of VF wetlands relates to the intermittent feeding regime that results in a series of daily batch cycles whereby influent feed is dosed onto the surface of a VFW over the course of several minutes and then allowed to drain, remaining unsaturated until the next feed dose. During the feed and drain portions of the cycle, contaminants within the feed adsorb into the biofilm for treatment during the unsaturated portion of the cycle (Paing et al, 2015). As the feed is dosed onto the surface of the VFW, it traps the air within the media which is then pushed through the depth of the wetland as the water percolates downwards (Stefanakis et al, 2014). Subsequently, as the water drains through the VFW, atmospheric air is drawn into the media through convection. Once the air is stationed within the media pores, it is able to diffuse through the wetted boundary layer and into the biofilm, where nitrification can occur (Molle et al., 2006). Additionally, the air within the VFW media during the rest period between feeds promotes oxidation processes contributing to mineralisation and degradation of retained solids helping to maintain hydraulic efficacy (Torrens et al., 2009; Vymazal, 2007). The effectiveness of the process is dependent on sustaining aerobic conditions such that when VFWs are used for raw or secondary treatment it is recommended that the beds are flooded for a maximum of 15.5hours per day (Arias Lopez 2013), limiting hydraulic loading rates to around 0.37m/d at dry weather flow (Morvannou et al., 2015; Troesch et al., 2014).

Overall, the daily dosing frequency (DF) applied to the VFW has a significant impact on the treatment capacity and operational stability of the bed and hence needs to be optimised to the given application. Low daily dosing frequencies are associated with an increased individual batch volume which increases the velocity of the water through the media, reducing contact time between the biofilm surrounded media and wastewater. This action promotes oxygenation within the bed, but decreases the long term nitrification capacity due to restricted contact time (Molle et al., 2006). Conversely, a high daily dosing frequency restricts time between feeds for re-oxygenation reducing mineralisation and degradation of trapped solids. Further, high dosing frequencies have been associated with less even biofilm growth through the depth of the bed with the majority occurring within the top 30cm once the DF equals 8 (Bancolé et al., 2003). The impact is an increased risk of clogging and hydraulic overloads which may expand the required design footprint (Molle et al., 2006). For instance, in the case of whole and secondary systems, the VFWs are known to be oxygen limited, as opposed to load limited (Molle et al., 2006; Torrens et al., 2009) such that it is common to operate with low daily dosing frequencies of between 3 and 10 feeds per day (Stefanakis et al, 2014).

Vertical flow wetlands are uncommonly used as a tertiary treatment with a commensurate paucity of available literature. The few available studies to date indicate similar DFs to those used for upstream operations at between 4 and 12 feed doses per day (Cooper et al, 1997; Schönerklee et al., 1997). However, the application of VFWs for tertiary treatment poses a different set of feed characteristics where the solids, organic and nutrient loads are relatively weak and there is a high dissolved oxygen status, enhancing conditions for nitrification reactions. Accordingly, it is posited that the systems are more likely to be load limited and hence will benefit from a higher DF to maximise adsorption of the ammonia. The current study aims to explore this by determining the impact of daily dosing frequency, ranging between 4 and 45 doses per day, on the treatment performance and clogging potential of VFWs used for tertiary treatment. The work encompasses the use of six pilot VFWs operated on an active sewage works and compared the nitrification performance to existing alternative tertiary treatments on the site. From this a recommendation will be made for the optimum dosing frequency for tertiary treatment VFWs, which will consider a balance between treatment performance and clogging potential

5.2 Materials and Methods

5.2.1 Site Description

The study was conducted on a wastewater treatment works in the Midlands, UK, designed with a treatment capacity of 65,000 population equivalents (PE). Onsite treatment was performed by a series of primary settlement tanks, trickling filters and humus tanks for secondary treatment and settlement followed by both tertiary nitrifying sand filters (SF) and nitrifying submerged aerated filters (NSAF), each of which receives 50% of the humus tank effluent (Figure 5-1). The site operates to achieve numeric discharge consents (95th percentile) of 25mgSS/L of suspended solids, 10mgO$_2$/L of biochemical oxygen demand, 3mgNH$_4$-N/L of ammonium-nitrogen and 2mgTP/L of total phosphorus. Total phosphorus removal was enhanced by chemical dosing of ferric sulphate and lime, pre and post secondary treatment.

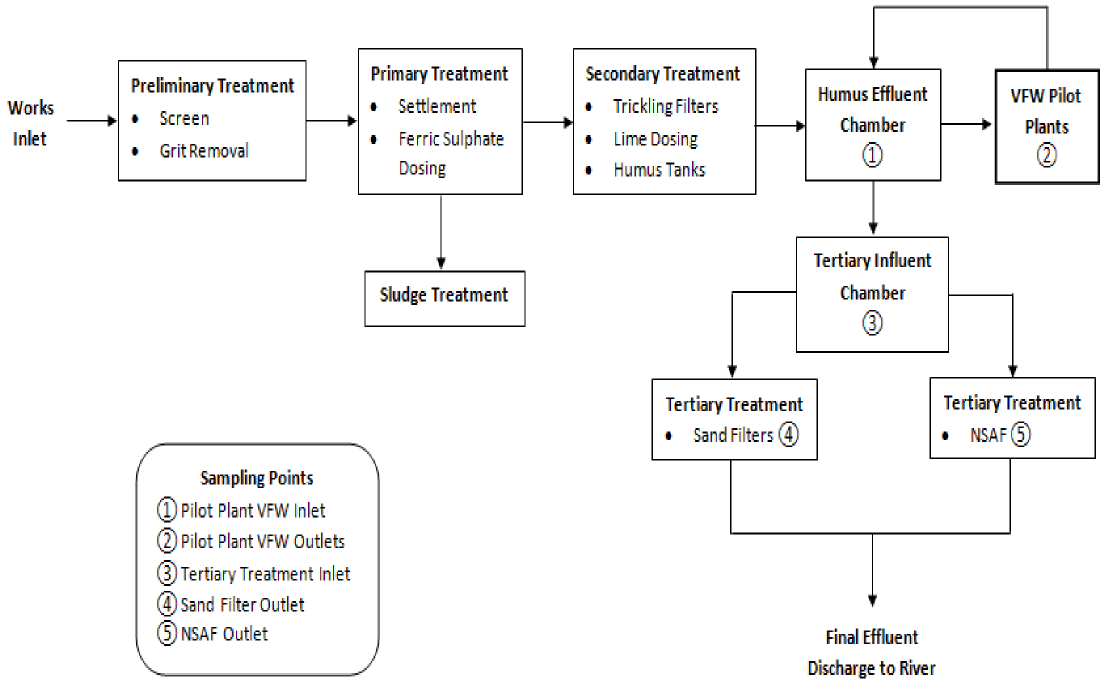


Figure 5-1 Process flow diagram of the wastewater treatment works including sampling points used for this study.

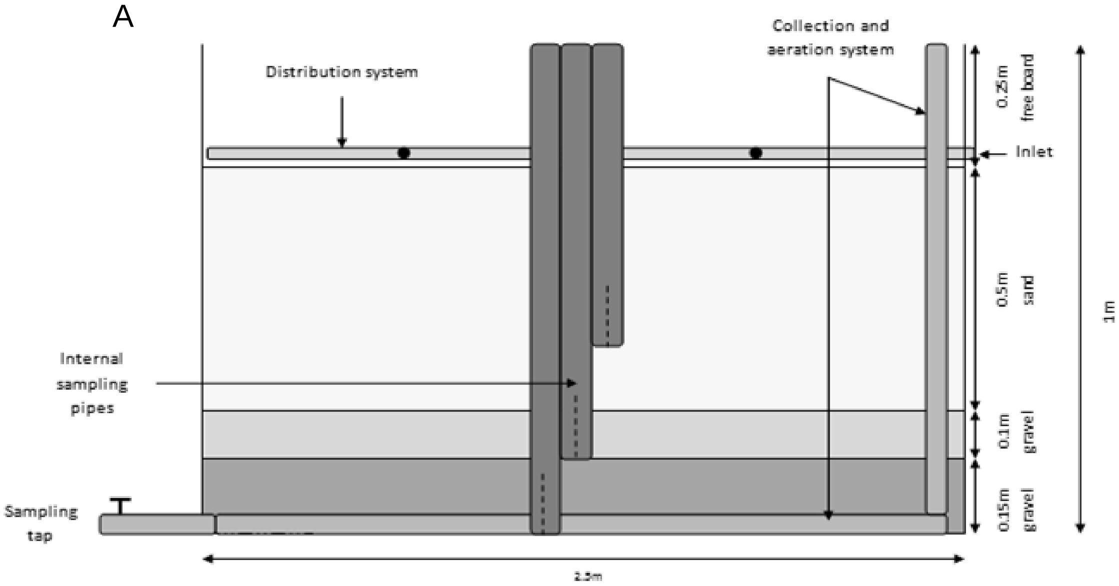
5.2.2 Experimental Design

Six pilot plant VFWs were used for the study and were positioned onsite to receive the humus tank effluent (Figure 5-1). To feed the VFWs, wastewater was continuously pumped from the humus effluent chamber to the VFW header feed tank which could then be pumped to the individual VFW pilot plants. Surplus wastewater from the VFW header feed tank was redirected back to the humus effluent chamber via an overflow system, ensuring a continual fresh wastewater supply to the VFWs.

Each of the six VFW pilot plants totalled a surface area of 6.25m^2 per bed. The VFWs were constructed to a total depth of 1m, comprising the following (from top to bottom): 0.25m freeboard to accommodate potential hydraulic loadings or clogging events; 0.5m of ≤4mm ø concreting sand for main treatment; 0.1m of 4-8mm ø pea gravel for transition; and 0.15m of 16-32mm ø cobble stones for drainage. The pilot plants included perforated and capped sampling pipes within each of the media layers to enable collection of internal water samples (Figure 5-2A). Each VFW received a hydraulic loading rate (HLR) of 0.4m/d, delivered in an intermittent feeding regime of 5 minute pumped batch feeds followed by a prolonged drainage period. Different daily dosing frequencies (DF) were applied to the separate VFWs to establish the impact of operation on the performance. The daily dosing frequencies applied during the trial were 4, 8, 18, 24, 32 and 45 feeds per day (From here on in known as DF4, DF8, DF18, DF24, DF32 and DF45). All six of the VFWs were planted with *Phragmites australis* during August 2014 to allow for establishment prior to commencement of the experimental work (Figure 5-2B).

The study was conducted between October 2014 and March 2015, for a total of 175 days. Autumn and winter in the UK are typically the coldest and wettest seasons, with average monthly air temperatures during the experimental period ranging between 3.5°C and 12.6°C, and average monthly rainfall between 32.4 and 70.8mm. The water temperature through the pilot plants did not fall below 8°C during the experimental period. The VFWs were commissioned in July 2013 and had been in operation for 14 months prior to this study. Filter flushes with potable water were conducted to return the media to a clean state prior to the commencement of the current trial.

A



B

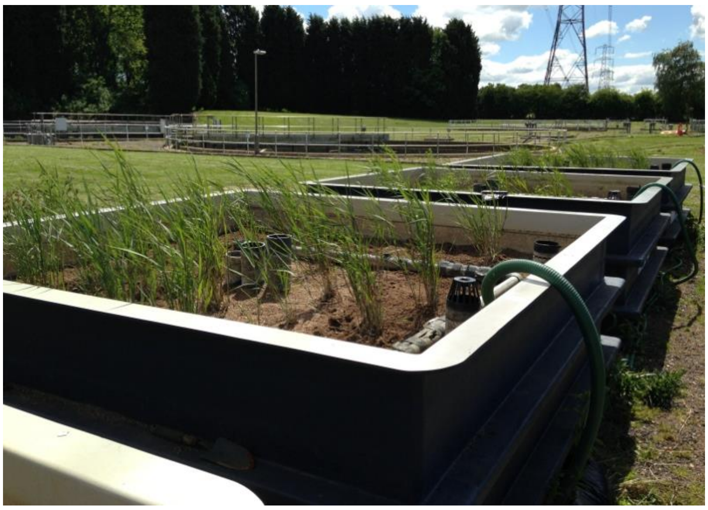


Figure 5-2 (A) schematic of the pilot plant VFW design used in the study. (B) Photo of pilot plant VFWs showing initial reed development prior to trial commencement.

5.2.3 Water Sampling and Analysis

Grab samples were collected from the influent and effluent of the VFWs on a weekly basis, providing a total of 25 sampling campaigns during the experimental period. Additionally, wastewater samples were collected from the onsite tertiary treatment influent and from the effluent of both the sand filter and NSAF (*n=18*), to provide a comparison of performance. The influent and effluent samples were collected in 1 litre plastic sampling bottles are were transported directly to the laboratory for same day analysis. The tertiary treatment influent sampling point was downstream of the VFW influent feed, and as such slight variations in the influent pollutant characteristics were observed (Table 5-1). Water samples from within the internal sampling pipes were collected periodically (*n=7*) and transported to the laboratory for same day analysis. Due to the small sample size from within each of the internal sampling pipes (<30ml), laboratory analysis was limited to the use of test kits and probe readings. Influent drainage time through each of the VFWs was monitored using a level sensor (levelogger EDGE model 3001, Solinst Canada LTD) placed within the drainage layer internal sampling pipe.

Influent, effluent and internal water samples were analysed for ammonium-nitrogen (NH_4-N), nitrite-nitrogen (NO_2-N), nitrate-nitrogen (NO_3-N), total phosphorus (TP) and chemical oxygen demand (COD) concentrations using colourimetric test kits, as per the manufacturer's instructions (Hach, Germany). Alkalinity analysis was performed using a drop count titration method (Hach, Germany). The dissolved oxygen concentration (DO), pH and redox potential (ORP) were measured using probes (LDO sensor, pH gel electrode and ORP electrode; Hach, Germany) and a portable multi-meter (HQ40d multi-meter; Hach, Germany). Total suspended solids (TSS) and volatile suspended solids (VSS) were analysed according to standard methods (APHA, 2005), using filters with a particle retention size of 1.2µm. A 5-day carbonaceous biochemical oxygen demand ($cBOD_5$) was conducted periodically *(n=10)* using BOD nutrient buffer pillows and nitrification inhibitor, according to the manufacturers guidelines (Hach, Germany). Influent and effluent samples for metals analysis were preserved by filtering 30ml through a 0.45µm pore sized membrane and adding

3ml of concentrated trace metal grade nitric acid, before being stored at 4°C until analysis. Metal analysis samples were analysed for nickel (Ni), zinc (Zn), lead (Pb), copper (Cu), Cadmium (Cd), and chromium (Cr) using inductively coupled plasma-mass spectrometry (ICP-MS) and Iron (Fe) using flame atomic absorption spectroscopy (FAAS), and was conducted on a two monthly basis.

Table 5-1 Mean VFW and tertiary treatment Influent characteristics (and concentration range).

Wastewater Parameters	Wetland Influent (mg/L)	SF/NSAF influent (mg/L)
TSS	27.0 (15.0-46.0)	27.2 (17.3-39.7)
VSS	17.3 (11.0-29.3)	17.0 (10.7-24.7)
COD	58.1 (28.3-83.4)	61.6 (27.4-85.8)
NH_4-N	3.8 (1.2-6.4)	3.7 (1.2-6.4)
NO_2-N	0.38 (0.26-0.47)	0.35 (0.25-0.47)
NO_3-N	27.1 (14.0-33.7)	25.9 (14.2-33.4)
TP	1.22 (0.75-1.81)	1.26 (0.68-1.78)
DO	7.6 (5.9-9.8)	7.5 (5.6-9.5)
pH	7.2 (6.7-8.1)	7.1 (6.7-7.8)
ORP	195.6 (24.7-378.3)	175 (42.2-395.1)
Alkalinity	54.2 (30-80)	55.5 (40-80)

Membrane fractionation on VFW influent and effluent samples was conducted on a monthly basis in order to differentiate between total phosphorus and metal concentrations within the particulate, colloidal and dissolved fractions of the samples. For this a nitrogen gas supply was used to maintain an atmospheric pressure of 3.7 (atm) within an Amicon 8400 stirred pressure cell (Millipore, UK). Polyethersulfone membranes of pore size 100kDa and 1kDa (Omega™, PALL life Sciences, UK), were used to separate the particulate-colloidal fractions and the colloidal-dissolved fractions, respectively. Prior to, and between uses, the membranes were prepared and cleaned using a wash sequence of ultrapure water, 0.1M sodium hydroxide (NaOH) and 0.1M hydrochloric acid (HCl) (McAdam et al., 2007).

5.2.4 Statistical Analysis

Minitab (Version 17) statistical software (Minitab Inc, Pennsylvania, USA) was used to calculate one way ANOVA (analysis of variance), to determine significant differences in removal efficiencies between wetlands receiving different dosing frequencies, using a significance level of $\alpha=0.05$. Pearson product-moment correlation coefficients and coefficients of determination were also used to determine the relationships between solids, organics and nutrient mass loadings and mass removals.

5.3 Results and Discussion

5.3.1 Effect of Dosing Frequency of Hydraulic Behaviour

Flushing the VFWs prior to commencement of the trials enabled all beds to operate under similar initial hydraulic character as observed through the infiltration rate and surface drainage time that varied between 7 to 9 minutes and 4 to 6 minutes respectively across all the beds. Infiltration rates of water through the VFWs were recorded using a levelogger and were defined as the time taken to reach the maximum height difference between the internal water levels during a feed. Similarly, drainage times were defined as the time taken for internal water levels to decrease from max height achieved during a feed to the minimum height.

Operation of the beds with different dosing frequencies altered the operational infiltration times from 17 minutes for a DF of 45 to 47 minutes at a DF of 4 (Table 5-2). Corresponding drainage times were 232 minutes and 13 minutes respectively, congruent with the difference in applied batch volume to each bed. To illustrate, the total volume applied per day was fixed at 2.5m^3, which was dosed in batch volumes from 55L (DF45) to 625L (DF4). The overall impact was to increase the contact time in the bed as the dosing frequency increased by reducing water velocity through the wetland (Molle et al., 2006; Torrens et al., 2009). The equivalent hydraulic retention time of the beds varied between 18 hours and 36 minutes at a DF of 4 and 22 hours, 30 minutes at a DF of 45, which were calculated as the sum of infiltration and drainage times in relation to the DF (Table 5-2).

Table 5-2 Infiltration, drainage and hydraulic retention time for each dosing frequency (data taken prior to clogging events).

	Internal water level increase (cm)	Infiltration time (mins)	Drainage time (mins)	HRT/Day (hours, mins)
DF4	3.15	47	232	18hrs 36mins
DF8	2.07	30	118	19hrs 44mins
DF18	1.03	25	38	18hrs 54mins
DF24	0.70	20	29	19hrs 36mins
DF32	0.50	20	18	20hrs 16mins
DF45	0.38	17	13	22hrs 30mins

Hydraulic overloading occurred at all dosing frequencies except DF4 which operated during the entire course of the trial without evidence of clogging consistent with findings from trials treating raw or secondary sewage (Molle et al, 2016). All other dosing frequencies resulted in hydraulic overloading that occurred within three months for beds dosed at frequencies of 18, 24 and 45 in comparison to events occurring within the fourth and fifth month for dosing frequencies of 32 and 8 respectively. In the case of VFWs used for treatment of raw sewage, which operate at a similar hydraulic loading rate to the current study, stable establishment of the systems is reported to take up to two years with limited reported data prior to that period (Chazarenc and Merlin 2005). A previous study during the first year of operation revealed similar hydraulic overload problems as observed here suggesting bed conditioning to be an important parameter. Interestingly, the lowest dosing frequency did not experience hydraulic overloading congruent with the better distribution and longer unsaturated periods associated with the larger, less frequent batch feedings.

The majority (9 out of 11), of the hydraulic overloading events occurred during periods of elevated solids (>10.8g/m^2/d) and COD loading (>23.3g/m^2/d) congruent with known causative links (Langergraber et al., 2003). The observed levels exceed reported limiting solids loading for single stage VFW treating secondary wastewater of 5gTSS/m^2/d (Winter and Goetz, 2003) but are below the limiting values that are reported to trigger non recoverable ponding (Langergraber et al, 2003). In the two stage “French” VFW systems used for treating screened raw sewage, the second stage beds are similar in media size to the current systems and operate with solids loadings up to 60gTSS/m^2/d (Troesch et al., 2014). However, the median solids loading rate was 10.8gTSS/m^2/d extending the operating range for single pass, no rotation VFW and indicating that higher solids loading rates can be used in tertiary VFWs. To illustrate, in the case of the VFW operated with a dosing frequency of 4, the maximum solid loading rate was 18.4gTSS/m^2/d and the bed never experienced hydraulic overloading. Overall, the two most stable beds were dosed at the extreme frequencies suggesting that DF was not a critical parameter in management of clogging within tertiary VFW.

To relieve the wetlands of the retained water, the surface of the wetland was perforated and left to drain freely before receiving additional feeds. The bed with the highest dosing frequency recovered after the intervention and suffered no further hydraulic overloading. Subsequent treatment analysis includes the full data set including the performance before and after hydraulic overloading. However, analysis assessing load limits was restricted to DF4 and DF45 as they experienced the most stable operation and hence provide the best assessment of load limits.

Influent to the VFWs had an average residual dissolved oxygen concentration of 7.6mgO_2/L, with a minimum measured DO of 5.9mgO_2/L. These concentrations were increased within the VFW effluent, with an overall average DO of at least 8.4mgO_2/L. In bed sample analysis showed DO concentrations to be between 7.6-9.3mgO_2/L within the main treatment layer (50cm depth), 7.8-8.6mgO_2/L within the transition media layer (75cm depth) and between 7.8-8.9mgO_2/L within the drainage layer (100cm depth). Therefore, DO is not considered a limiting factor for performance under normal conditions, during this study.

5.3.2 Effect of Dosing Frequency on Nitrification

During the initial 16 day start up phase the ammonia removal rate of the six VFWs converged to a rate of approximately 0.67gNH_4-N/m^2/d equating to a removal efficiency of 97% (Figure 5-3). Large variation was observed across the beds at the start with removal efficiencies between 41% and 98% with the higher initial removals associated with the low DF of 4 and 8. Other contaminant removal (COD, TSS and TP) were observed across all VFWs within 24 hours of experimental start up, with the exception of TSS removal in DF45. Within the initial 16 days of experimentation, TSS remained higher within the effluent of DF45, than was input within the influent, and is likely to have occurred as fallout of solids resulting from the VFW filter flush. This continued to influence the solids content within effluent samples into the fourth month of the experimental period (Figure 5-4).

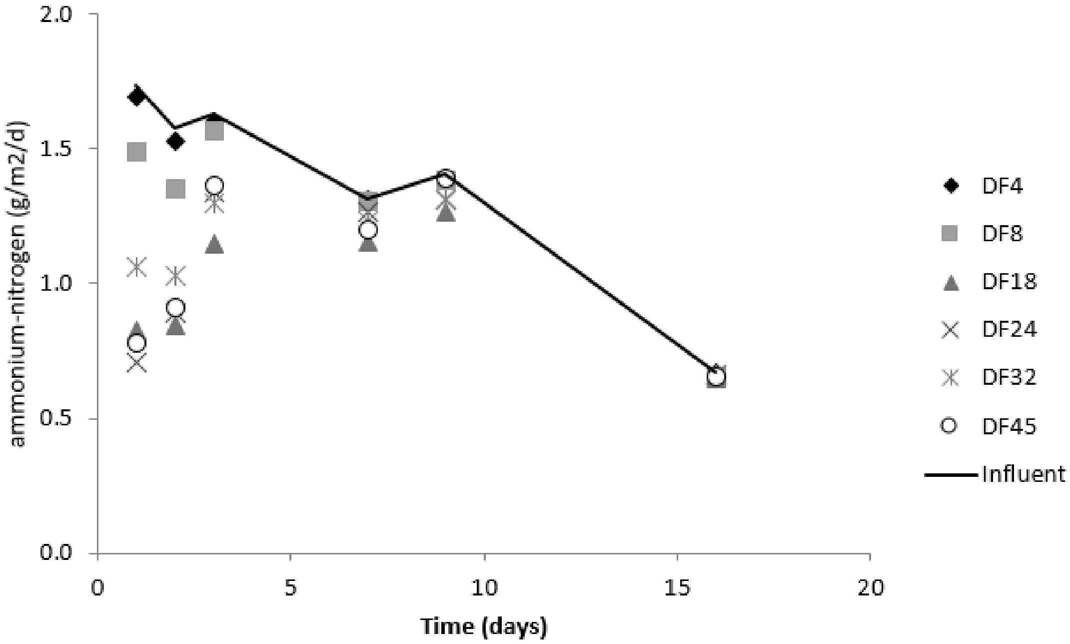


Figure 5-3 Ammonium-nitrogen removal rates within the initial 16 days of VFW operation.

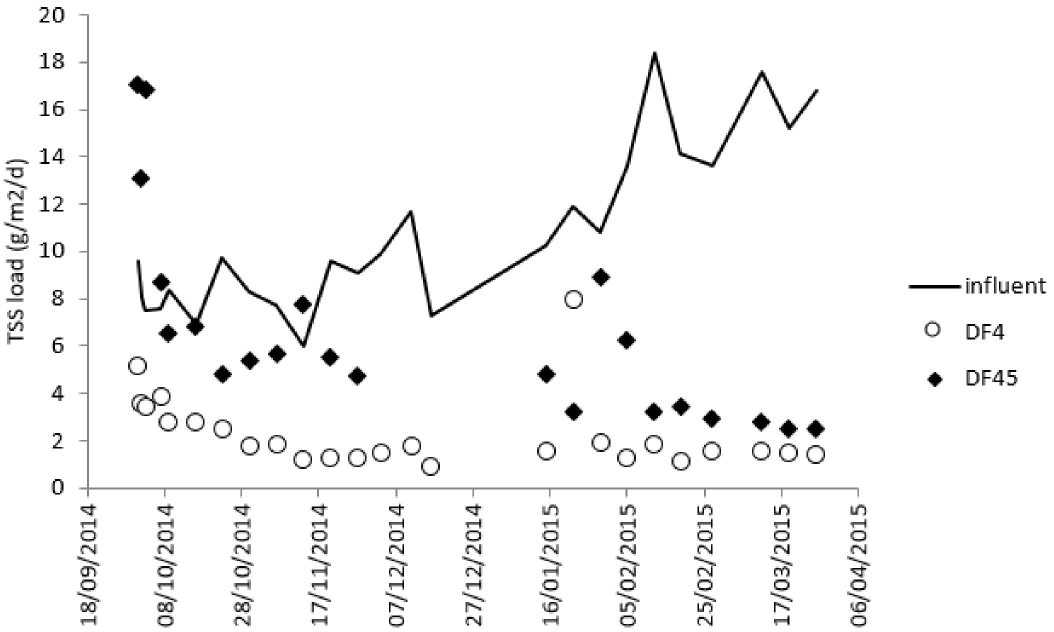


Figure 5-4 Timeline of total suspended solids 'fallout' in DF4 and DF45, for the duration of the experimental period.

Across the trial period the average ammonium-nitrogen concentration decreased from 3.8mgNH_4-N/L in the influent to between 0.03 and 0.57mgNH_4-N/L (excluding outliers due to clogging) post the VFWs operating with the different dosing frequencies (Figure 5-5). Comparison of the different dosing frequencies revealed better performance with lower dosing frequencies. To illustrate, the removal efficiency reduced from 99% at a DF of 4 to 86% at a DF of 45. In terms of DF4, a nitrogen mass balance indicated a mean ammonium-nitrogen removal of 9.43gNH_4-N/d, with an associated nitrate-nitrogen increase of 11.29gNO_3-N/d. Equivalent numbers for the higher dosing frequency were 8.16gNH_4-N/d removed and 7.15gNO_3-N/d produced. Consequently, the VFWs are nitrifying with the differences attributed to a combination of the high initial nitrate concentration and the potential for adsorption-flushing across cycles.

All the VFWs experienced hydraulic overloading events during the trials with the exception of DF4 and this affected subsequent treatment efficiency and nitrification capacity. The impact of these clogging events was to restrict oxygen within the bed resulting in a decrease in nitrification (Chapter 3). For instance, in the case of DF8, a clogging event occurred six weeks before the end of the study, and whilst the surface ponding was remediated the VFW never fully recovered with an increase in effluent ammonium-nitrogen from 0.12mgNH_4-N/L prior to clogging to 2.75-6.08mgNH_4-N/L afterwards (Figure 5-5). Overall, across the entire trial the removal efficiency of the beds was 70%, 68%, 84% and 58% for DF8, DF18, DF24 and DF32 respectively. Excluding the post clogging events the equivalent removal efficiencies were 97%, 85%, 90% and 90% respectively demonstrating the importance of clogging management to the successful operation of the VFWs. The VFW operating at a DF45 was also subjected to one clogging event, however the surface remained flooded for three consecutive weeks, despite attempted remediation. Samples could not be obtained during this three week period, due to infiltration from the systems overflow. After the three week period, the VFW was able to drain down and VFW ammonium-nitrogen performance efficiency was returned, achieving effluent concentrations of below 1.3mgNH_4-N/L thereafter. Similar impacts of DF has previously been reported when using VFWs for raw and secondary wastewater treatment (Bancolé et al., 2003; Molle et al., 2006; Torrens et al., 2009). In such cases, the benefits of the low DF is associated to increasing the unsaturated period of operation as the systems are known to be oxygen limited. However, the dissolved oxygen concentrations remained high across all DFs unless hydraulic overloading occurred and as such the systems are postulated to be load rather than oxygen limited. In such cases the lower DFs provide the feed in larger batch volumes and it is posited that this enables better establishment of an active nitrification community within the biofilms. To explore this suggestion, a series of nitrification assays were taken to quantify the number of nitrifiers and the overall nitrification activity levels within the biofilms. Unfortunately, the assays did not generate reliable data and so it remains unclear as to the reason why lower DFs are more effective in the case of tertiary VFWs. Alkalinity concentrations increased between the VFW influent and effluent in all VFWs, with increases of between 78.5-94.6mg/L (145-175% increase between the influent and effluent). The observed increase is beyond that attributable to denitrification and as such, it is assumed that adsorption is occurring from the gas phase. Nitrification is believed to be inhibited at alkalinity levels of below 50mg/L, with 8.6mgHCO_3 required per 1gNH_4-N for successful nitrification (Cooper et al., 1996). Although the average alkalinity to the VFWs was 54.2±13.5mg/L, with influent alkalinity ≤50mg/L on 11 separate occasions, the increase in alkalinity during treatment prevented inhibition of nitrification from occurring.

Comparison to the onsite tertiary nitrification processes revealed lower residuals in the case of the VFWs (Figure 5-5). To illustrate, the ammonium-nitrogen decreased from an average influent concentration of 3.68mgNH_4-N/L to 0.83mgNH_4-N/L and 1.05mgNH_4-N/L for the sand filter (SF) and nitrifying submerged aerated filter (NSAF), respectively. The corresponding removal efficiencies were 78% and 80% respectively confirming the superior removal observed in the VFWs. The alkalinity within both the SF and NSAF decreased between the influent and effluent, as is expected during nitrification and indicates a potential risk to limited alkalinity in the wastewater.

The ammonium-nitrogen removal rate in the VFWs increased with the influent loading rate and exhibited a linear relationship up to an ammonium-nitrogen loading of 2.5gNH_4-N/m^2/d (Figure 5-6). Within the individual VFWs, Strong correlations were observed between ammonium-nitrogen loading and ammonium-nitrogen load removal in DF4 ($r=1.0$, $p=0$, $R^2=99.92\%$) and DF45 ($r=0.910$, $p=0$, $R^2=82.8\%$). Statistical analysis (one-way ANOVA, where $p<0.05$ shows a significant difference) determined there was a significant difference ($p=0.005$) in the ammonium-nitrogen mass removal efficiencies between the VFWs with different dosing frequencies, inclusive of data obtained during and post hydraulic overloading events. The current maximum effective loading rate was limited by mass loading suggesting higher loads would be possible if the feed concentration was higher. The current levels exceed those reported previously for tertiary VFWs at 0.4gNH_4-N/m_2/d (Schonerklee et al, 1997) but are less than the 10gNH_4-N/m_2/d reported for aerated HFWs (Butterworth et al, 2013) based on spot sampling during a full scale trial. The equivalent average load was 3.1±2.4gNH_4-N/m_2/d which is similar to the levels reported here. In both cases, ammonia loading rates do not appear to be the rate limiting component. Instead, in these dilute wastewaters, the hydraulic loading rate appears to be the rate limiting factor.

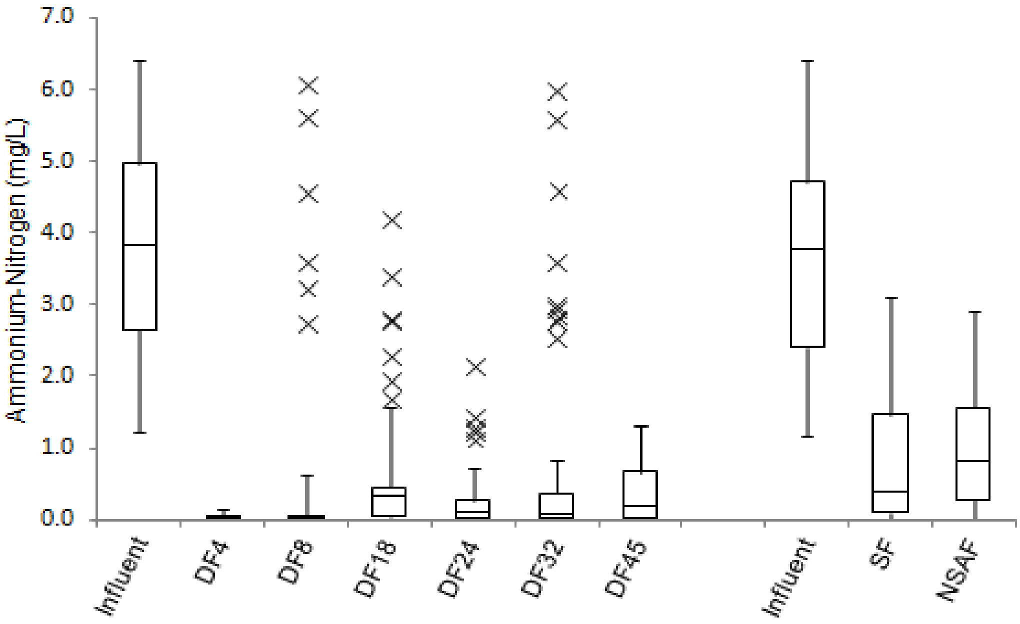


Figure 5-5 Box and whisker plot showing ammonium-nitrogen concentration removal in all VFWs. Data post clogging events are shown as outliers. Average influent and effluent ammonium-nitrogen concentrations for the onsite tertiary treatment are included for comparison (*n* = 18). The box represents the median, upper and lower quartiles; the whiskers represent the minimum and maximum; the crosses represent the outliers.

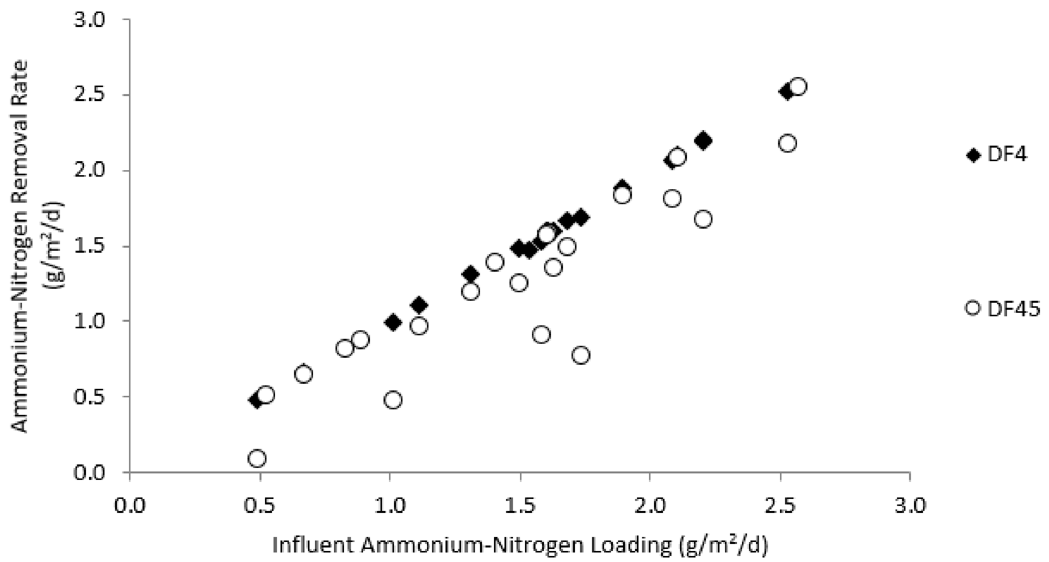


Figure 5-6 Ammonium-nitrogen loading and associated removal rate for the VFWs receiving the highest and lowest dosing frequencies.

5.3.3 Effect of Dosing Frequency on Solids and Organics Removal

The VFW were effective at removing residual organics with the mean COD reducing from an influent concentration of 58.2mgCOD/L down to a range between 25.0 and 18.7mgCOD/L across the six VFWs. In comparison, much lower removal of COD was observed across the SF and NSAF with average effluent levels of 37.4mgCOD/L and 46.0mgCOD/L (Figure 5-7 A&B). These levels correspond to removal efficiencies between 57% and 68% for the VFW compared to 25% and 39% for the NSAF and SF respectively. Greater variation was observed with DF above 4 reflecting the hydraulic overloading events that took place. Statistical analysis, confirmed by one-way ANOVA, showed a significant difference in COD load removal ($p<0.05$) as a function of dosing frequency. Strong linear relationships were observed at the two DF extremes (DF4: $r=0.908$, $p=0$, $R^2=82.5\%$; and DF45: $r=0.802$, $p=0$, $R^2=64.3\%$). Five day carbonaceous biochemical oxygen demand ($cBOD_5$), conducted periodically through the experimental period (*n=10*), had an average influent concentration of 5.5mg/L and a range of 3.4-6.2mg/L. Dissolved Oxygen did not have a significant decrease within the five day period to produce reliable results. Effluent samples without dilution (of nutrient buffer) had $cBOD_5$ concentrations ranging between 2.2-2.8mg/L, equating to efficiencies of 49-60%. As observed in terms of nitrification, the systems do not appear to be organic load limited as expected of tertiary treatment technologies.

The VFW also provided effective management of suspended solids with average residual suspended solids concentrations between 4.8 and 9.5mgTSS/L across VFW operating with DF between 4 and 32. Poorer removal was observed at the highest DF with an average residual suspended solids concentration of 16.3mgTSS/L. This equates to a removal efficiency of 40% compared to the other VFW where removal efficiency varied between 65% and 82%. Equivalent data for the NSAF and SF were average residuals of 16.6mgTSS/L and 9.2mgTSS/L respectively representing removals of 39 and 66% (Figure 5-7 C&D). The poorer performance of DF45 can be, in part, attributed to the initial flush of solids that were generated during the pre-trial flush although the solids concentration remained higher in DF45 compared to DF4 across the entire trial. Despite the poor TSS removal by DF45, a strong correlation was observed between influent TSS loading and load removed (r=0.891, p=0, R^2=79.4), even including initial results with an increased effluent load (r=0.798, p=0, R^2=63.7). Additionally, a strong correlation was seen between TSS removal and time (r=0.834, p=0, R^2=69.5) which indicated the VFW was recovering with time. Analysis showed VSS accounted for 64% of the TSS within the influent, and had various influence on the effluent solids content ranging from 28% in DF45 to 67% in DF32.

The removal of total phosphorus varied between 62 and 78% with average residual concentrations between 0.28 and 0.46mgTP/L (Figure 5-7 E&F). In comparison, removal of TP across the onsite technologies was much poorer with residual concentrations of 0.61mgTP/L and 0.91mgTP/L for the SF and NSAF, respectively. The reduced removal is congruent with the larger porosity media (3-4mm) used for both technologies compared to the 0-4mm used for the VFWs. Analysis during the membrane fractionation of VFW TP suggested the greatest removal was observed within the particulate fraction, with removals between influent and effluent particulates ranging from 0.71mgTP/L to 0.91mgTP/L, equating to efficiencies of between 71-92% (Figure 5-8). Removal from within the colloidal fraction remained low throughout, apart from within DF4 which has a removal of 0.047mgTP/L (64%). Removal was lowest across the dissolved fraction at between 4-31% reflecting that the major removal pathway was capture of solids rather than dissolved phosphorus adsorption. Overall strong correlations were observed between TP loading and load removed for both DF4 and DF45 (DF4: r=0.973, p=0, R^2=94.7%; and DF45: r=0.937, p=0, R^2= 87.8%) with no statistical difference observed between the two (p=0.250) suggesting DF was not an influencing factor for TP removal.

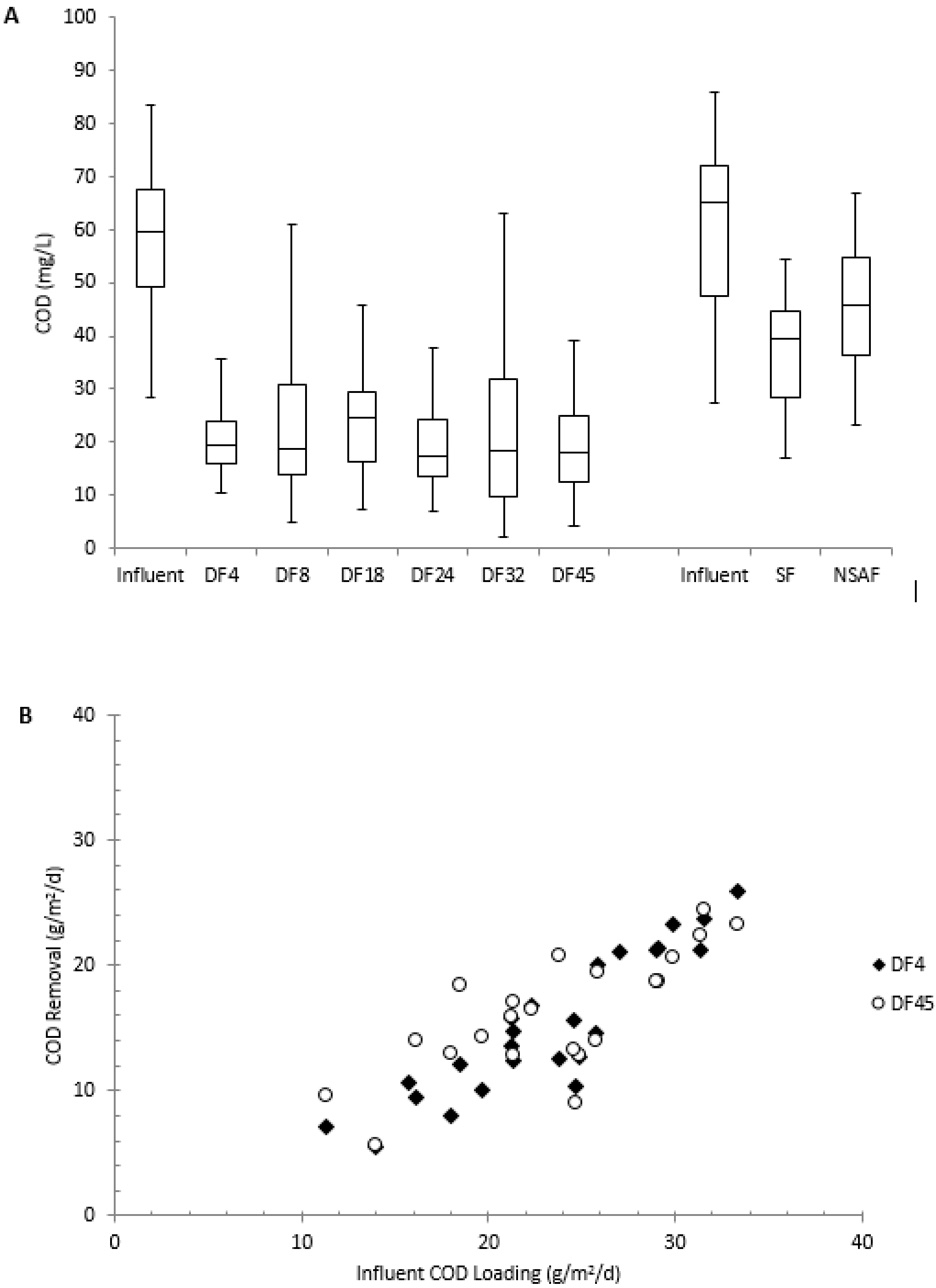


Figure 5-7 (A) Box and Whisker plot showing the Influent and effluent COD concentrations of the VFWs and the comparison tertiary treatments. The box represents the median and the upper and lower quartiles and the whiskers show the maximum and minimum concentrations. (B) COD loading and associated removal rate for the VFWs receiving the highest and lowest dosing frequencies.

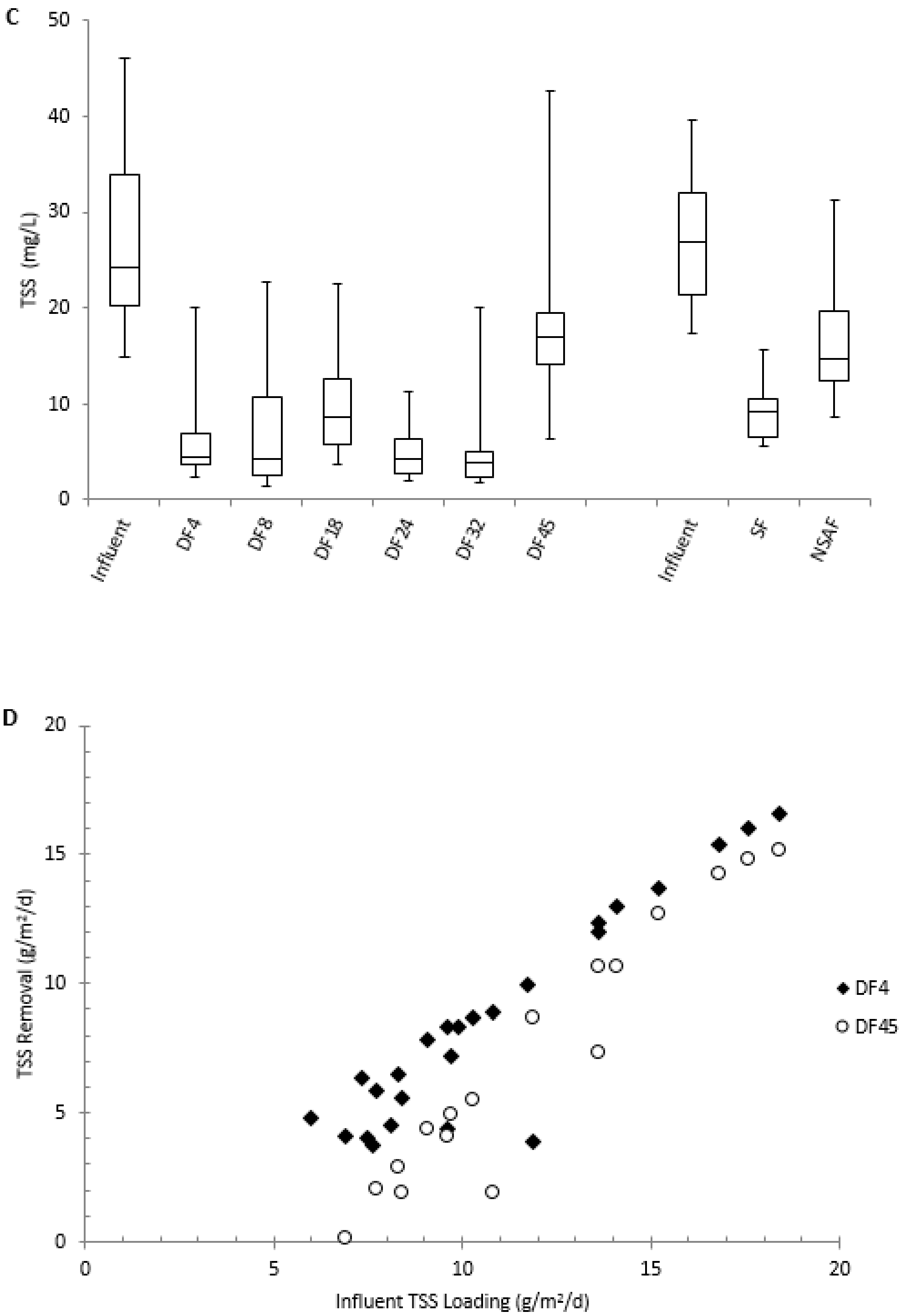
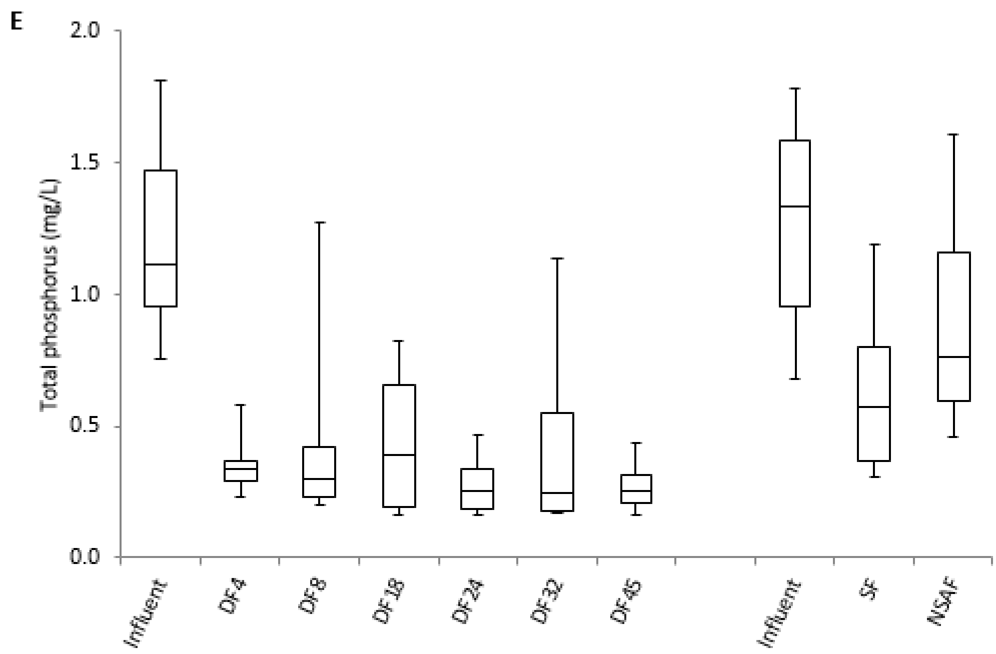


Figure 5-7 C) Box and Whisker plot showing the Influent and effluent TSS concentrations of the VFWs and the comparison tertiary treatments. The box represents the median and the upper and lower quartiles and the whiskers show the maximum and minimum concentrations. D) TSS loading and associated removal rate for the VFWs receiving the highest and lowest dosing frequencies.



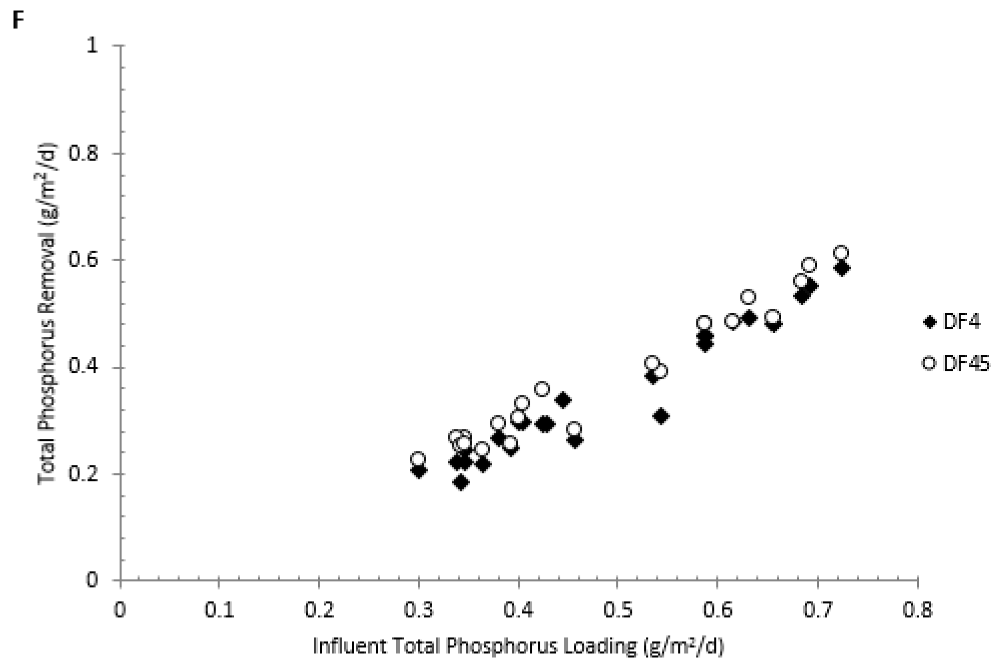


Figure 5-7 E) Box and Whisker plot showing the Influent and effluent TP concentrations of the VFWs and the comparison tertiary treatments. The box represents the median and the upper and lower quartiles and the whiskers show the maximum and minimum concentrations. F) TP loading and associated removal rate for the VFWs receiving the highest and lowest dosing frequencies.

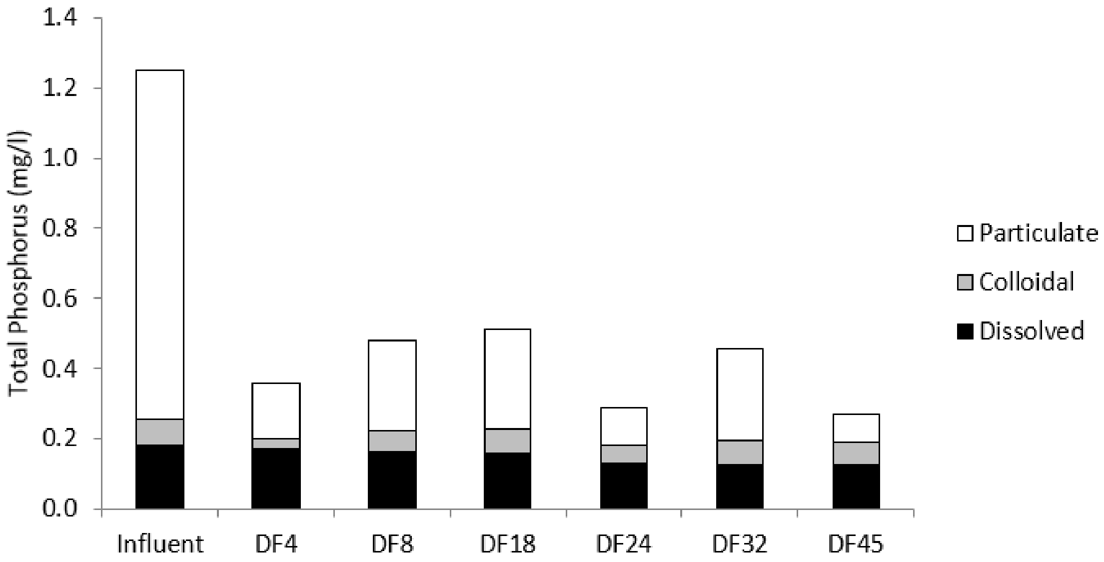


Figure 5-8 Fractionation of TP, showing particulate, colloidal and dissolved fractions.

Heavy metals analysis showed a presence of iron, zinc, copper and chromium within the influent to the VFWs, with concentrations of 1.7µgFe/L, 48.1µgZn/L, 15.7µgCu/L and 4.0µgCr/L, respectively. Determinands for analysis also included lead, nickel and cadmium, however influent and effluent concentrations remained below the ICP-MS limit of detection (1µg/L). Iron removal was the most successful, achieving efficiencies of between 99.5% and 100%, and producing effluent iron concentrations of <0.06µg/L within all the VFWs. Chromium and copper removal was also successful across all VFWs with effluent concentrations of between 2.1-2.8µgCr/L (94-96%) and 7.3-8.7µgCu/L (82-85%), respectively. Zinc removal was also observed across all VFWs, although at a lower efficiency, producing effluent concentrations of between 34.7-38.8µg/L equating to efficiencies of between 19-28%.

5.4 Conclusions

The efficacy of using VFW for tertiary polishing of wastewater effluents for ammonia removal has been demonstrated. Comparison with established technologies such as sand filters and nitrifying submerged aerated filters show that VFW are able to match and even improve residual concentrations of ammonia, COD, solids and phosphorous. The systems appear to be hydraulically limited rather than load or oxygen limited. Loading rates trialled were higher than standard practice for VFW including systems utilising rotation to manage solids. Whilst the best performance was observed with the lowest dosing frequency, in terms of hydraulic and treatment performance no clear relationship could be established. In fact, similar performances were observed at the two ends of the dosing frequencies trialled suggesting that DF is not a critical design choice when considering tertiary VFWs. This is in clear contrast to application of the technology for raw and secondary applications were DF are kept low to ensure effective treatment.

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Chapter 6

Impact of resting periods on the hydraulic behaviour and performance potential within tertiary vertical flow wetlands

6 Impact of Resting Periods on the Hydraulic Behaviour and Performance Potential with Tertiary Vertical Wetlands

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Abstract

Clogging can have a detrimental effect on the hydraulic conductivity and performance potential in vertical flow wetlands (VFWs). For whole treatment and secondary VFWs, the application of prolonged resting between feeding periods have shown positive results regarding the prevention of clogging occurrence and remediation of clogged systems. Due to the limited operational recommendations within the literature for tertiary VFW systems, the role of resting periods is unknown for this application. This study aims to explore the potential to operate VFWs for tertiary treatment at elevated hydraulic loading rates (HLR) through the use of resting periods and different dosing frequencies on stabilised pilot plant VFWs. Performance across all VFWs showed removal efficiencies of between 72-83% for solids removal from influent concentrations of 20±3.98mgTSS/L, 57-73% for COD removal from influent concentrations of 53±5.54mgCOD/L, 74-100% for ammonium-nitrogen from influent concentrations of 2.81±1.39NH_4-Nmg/L and 55-72% for total phosphorus from influent concentrations of 1.09±0.18mgTP/L, showing clear potential for achieving concentrations within proposed effluent discharge consents. Performance, however, was not significantly influenced by either instantaneous HLR, resting periods or dosing frequency, suggesting that optimal operational limits had not been reached. Progression of hydraulic acceptability within the VFWs over a (previous) 20 month experimental period has shown a step wise increase of maximum operable

HLR of 0.19m/d during the first experimental trial on an unplanted system (Chapter 4), to 0.4m/d during the second experimental trial (Chapter 5) and 0.8m/d during the current trial, whereby hydraulic limits were not reached. This shows a clear need for an initial stabilisation period of new VFWs of between 1-3 years, with indications leading towards potential future HLRs exceeding 1m/d.

6.1 Introduction

Sewage treatment works treating small populations (less than 2000 population equivalence), typically in rural locations, require a different set of attributes to the large urban wastewater treatment works. The future direction of the latter is towards resource recovery and alliance to the concepts of the circular economy (Stear, 2016). Whereas, for the former, the desired attributes align to concepts of “fit and forget” whereby the works use minimal external inputs of electricity and chemicals and require only occasional maintenance visits. Many existing works fit this paradigm but are being challenged by the onset of new requirements to treat nutrients such as ammonia and phosphorus down to low levels such as sub 3mg/L ammonia and sub 1 mg/L total phosphorus. Compliance puts a pressure towards the use of chemically and energy intensive technologies to reliably meet such standards, losing the original beneficial attributes and imposing significant infrastructure upgrading in terms of roads and facilities to accommodate the new options (Germain-Cripps, 2015). Accordingly, there is a desire to adapt the existing technology to meet the new requirements whilst not losing their near passive nature. This is perhaps best illustrated through constructed wetlands that are traditionally configured as subsurface horizontal flow systems to remove suspended solids and associated organics (Butterworth et al, 2016). Recent adaptions have seen inclusion of forced aeration systems altering the redox conditions within the bed to enable nitrification (Butterworth et al, 2013; Ouellet-Plamondon et al, 2006) as well swapping the media for reactive materials such as steel slag and apatite to enable enhanced phosphorus removal (Jefferson, 2016).

In relation to the need for increased nitrification capacity, the need for aerobic conditions can still be accomplished by utilising vertical rather than horizontal flow wetlands. Vertical flow wetlands (VFW) operate through periodic dosing of feed, which is then allowed to percolate through the bed. The beds are filled with finer media than those used for HFW, typically sand up to 4mm in diameter. The unsaturated nature of VFW provides the required aerobic conditions, increasing mineralisation rates as well as nitrification. The systems are typically applied to high load wastewaters such as raw (Molle et al, 2013) or secondary (Brix, 1994) treated in either single or two stage arrays. A recent survey of two stage systems, used extensively in France, reported typical effluent quality of 10±10mgTSS/L, 6±4mgO_2/L and 5±6mgNH_4-N/L for TSS, BOD_5 and NH_4-N respectively based on composite sampling (Paing et al. 2015). In contrast, there is a paucity of information pertaining to the use of VFW for tertiary treatment especially in relation of the need to meet more challenging discharge consents.

One of the main disadvantages associated with VFW is that they are susceptible to internal and surface clogging, which can limit treatment performances and reduce longevity of the system (Knowles et al., 2011). Previous studies have suggested that clogging potential can be reduced by optimising the operational strategies, including control of hydraulic loading and application of intermittent feeding regimes, and clogging occurrence can be alleviated by application of resting periods (Langergraber et al., 2003). For instance, current single stage systems operate with hydraulic loading rates of around 0.05m/d, which is increased to 0.37m/d when used in parallel to enable bed rotation (Morvannou et al., 2015; Troesch et al., 2014). Clogging is associated with the gradual accumulation of particulate solids and organic matter and production of the surplus sludge (Langergraber et al., 2003; Stefanakis et al., 2014; Vymazal et al., 1998). Accordingly, the potential to clog is a balance between the rate of loading of new solids and organics and the ability of the biofilm within the bed to process the load. Critically, the aerobic conditions must be maintained sufficiently to drive adequate mineralisation of the accumulating solids to stabilise the system. This is achieved through either low loading or the use of low dosing frequencies and rest cycles where the beds are left for several weeks to process the accumulated material (Molle et al., 2006; Torrens et al., 2009). For instance, the latter approach is commonly used within the French style, 2-stage rotational VFW system, which typically treats raw wastewaters with high influent solid and organic loading rates (Molle et al., 2005). The VFW receive intermittently fed wastewaters for 7 days followed by a resting period of 14 days. This prolonged rest time promotes dewatering of the accumulated surface solids, followed by microbial degradation and mineralisation processes to minimise volume of trapped solids, and reopen previously clogged pores. The systems operate at a HLR of 0.37m/d and are known to function for around 10 years between required refurbishments.

Previous work on tertiary VFWs examining the impact of HLR on unplanted beds (chapter 4) and different dosing frequencies (chapter 5) has shown that the systems operate differently to highly loaded VFW and that HLRs as high as 0.4m/d can be operated whilst delivering high levels of tertiary treatment. A key challenge with all VFW is that the hydraulic response of the system alters during the first 1-3 years of operation as the system stabilises (Gomez, 2016). It is hypothesised that tertiary VFWs with resting periods incorporated into normal operation will be more accepting of increased hydraulic loads, and subsequently less prone to clogging. Accordingly, the current paper aims to explore the potential to operate VFW for tertiary treatment at elevated HLRs through the use of resting periods and different dosing frequencies on pilot VFW that had been operating for 20 months and hence represented more stable systems.

6.2 Materials and Methods

6.2.1 System Design and Configuration

Eight vertical flow wetland (VFW) pilot plants were constructed onsite of a 65,000 population equivalent (pe) wastewater treatment works, in the midlands UK, and positioned to receive secondary treated effluent. Each of the pilot plants had a width and length of 2.5m, were 1m in height, and were constructed from glass reinforced plastic (Figure 6-1A). Each of the pilot plants comprised the following media configuration (from top to bottom): 0.5m of concreting sand (ø≤4mm); 0.1m of pea gravel (ø4-8mm); 0.15m large gravel (ø16-32mm). A freeboard of 0.25m was incorporated to accommodate potential hydraulic overloads. A distribution system, constructed from PVC pipe work, was installed on each of the pilot plants to provide a uniform distribution of water over the wetland surface. A drainage system, constructed from 10cm diameter PVC pipe work, was designed to mirror the distribution system and was positioned within the drainage media at the bottom of each wetland, to receive the treated effluent (Figure 6-1B). Overflows to accommodate hydraulic overloads were incorporated into the design of the drainage system and were positioned to reach 15cm above the media surface. Three sampling pipes were installed in the centre of each wetland to allow sampling from each of the media layers. The VFWs were planted with *Phragmites australis* at a density of 4 plants per m^2.

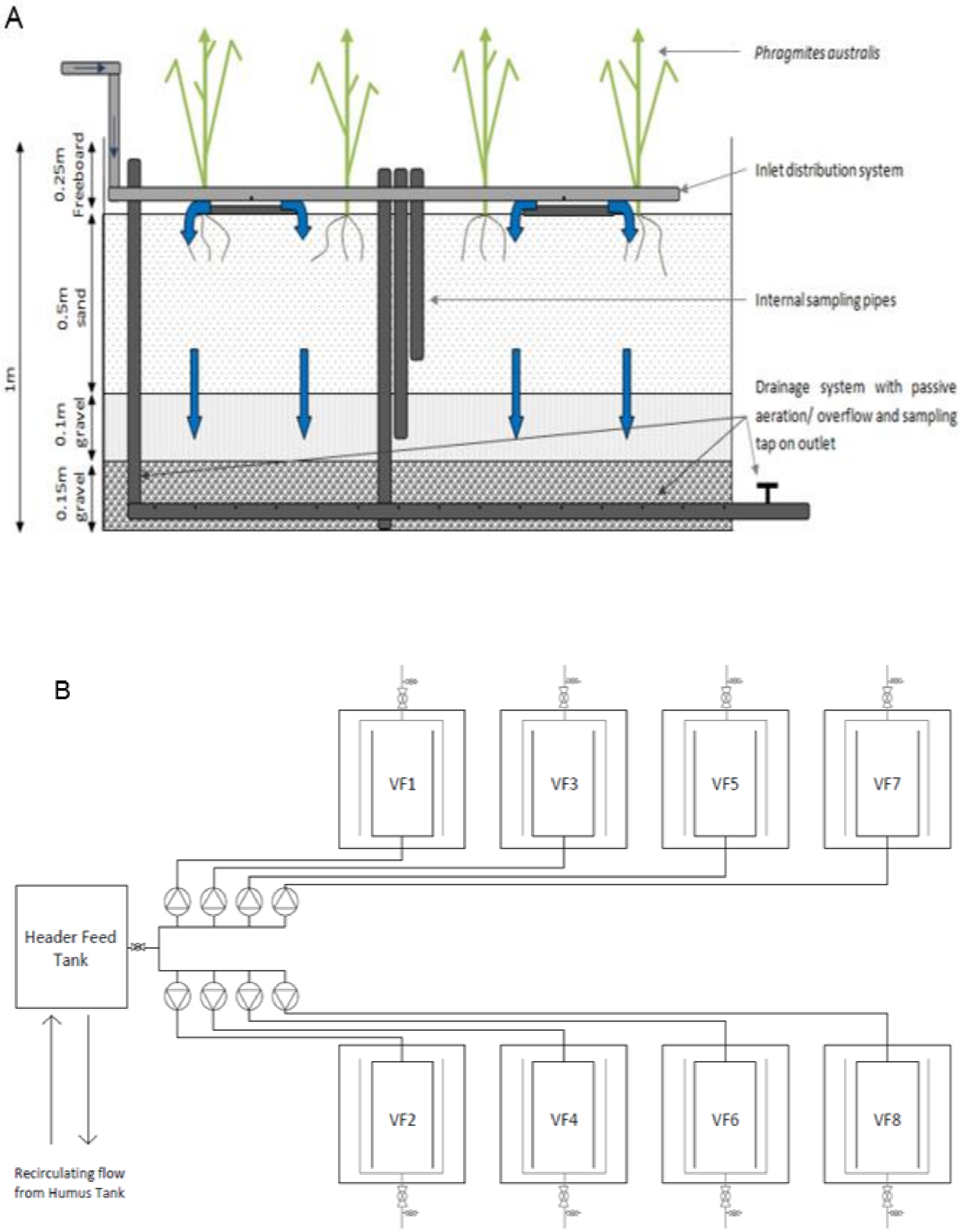


Figure 6-1 (A) Design of the pilot plant VFWs. (B) Areal view of the onsite VFW pilot plant configuration, showing the inlet distribution system and drainage system (light grey) and including VFW ID.

6.2.2 Experimental Prerequisites

The VFW pilot plants were constructed and commissioned onsite during July 2013 and were used in experimental trials for 20 months prior to this study. Prior to use in this study the VFWs were rested for one month to promote mineralisation and oxidation of the trapped solids and organic matter, before being flushed through with potable water. During the filter flush, the isolation valves on the outlet of the VFWs were shut, allowing the wetland to completely fill with potable water. Once filled, the outlet valves were opened and the wetlands were allowed to drain freely, with the water level drop and time taken to drain recorded using a level sensor with compensated atmospheric fluctuations (levelogger EDGE model 3001 and Barologger EDGE model 3001, Solinst Canada LTD) which was positioned within the drainage layer internal sampling pipe. Once flushed the VFWs were rested for a further two weeks before receiving a second filter flush using potable water, attempting to return all VFWs to a similar starting hydraulic conductivity (Table 6-1). The VFWs were rested a further two weeks prior to experimental commencement. The VFWs were planted with *Phragmites australis* in August 2014, which were fully established pre-commencing this study. During the initial month of rest, invasive plant species that had grown between the reeds on the wetland surface during the previous experimental period were removed.

Table 6-1 Pre-experimental filter flush times and operational strategies used throughout the experimental period.

ID	Flush time (minutes/seconds)			feed/ rest ratio	Daily dosing frequency	HLR during operation	Effective HLR
	1st Flush	2nd Flush					
VF1	6.34	3.27		4:3	4	0.8m/d	0.46m/d
VF2	3.36	3.18		5:2			0.57m/d
VF3	3.07	2.49		6:1			0.69m/d
VF4	2.41	2.18		7:0			0.80m/d
VF5	5.51	3.30		4:3	45		0.46m/d
VF6	3.36	3.24		5:2			0.57m/d
VF7	11.29	3.52		6:1			0.69m/d
VF8	12.41	3.41		7:0			0.80m/d

6.2.3 System Operation

The study was conducted between June and December 2015, with a total of 23 sampling campaigns over the six month period. Each of the pilot plant VFWs were fed using individual pumps that were programmed to control the hydraulic loading rate, daily dosing frequency and days of operation. As such the pilot plant VFWs were operated to receive a hydraulic loading rate (HLR) of 0.8m/d on a 'feeding' day, ensuring identical batch volume feeds within same dosing frequency groups, which averaged a HLR of between 0.46m/d and 0.80m/d over a week of operation (Table 6-1). Four different feed and rest strategies, ranging between no rest and 3 days rest over a 7 day period, were employed over two different daily dosing frequency regimes: 4 feeds per day and 45 feeds per day.

The greatest average monthly temperatures during the study were recorded for July and August (16°C) and the lowest for November and December (9°C). Recorded rainfall was lowest during September and October with total monthly rainfalls of <50mm, and the highest accumulated monthly rainfalls were recorded during August, November and December (87-115mm).

6.2.4 Hydraulic Conductivity

The hydraulic conductivity of each wetland was conducted onsite on a monthly basis, during an operation day of all VFWs to ensure a similar internal moisture saturation level, with measurements taken in duplicate, using an adapted method described by Pedescoll *et al.* (2009). A steel pipe, with a height of 1m and internal diameter of 0.105m, with perforations to a height of 0.25m was inserted 0.3m into the centre of the wetland surface, to ensure sufficient coverage of all perforations and to avoid preferential pathway and media washout in the shallower depths. The steel pipe was filled with 5L of wetland feed wastewater and the decrease in water level within the pipe was recorded at one second intervals using level sensor with compensated atmospheric fluctuations (levelogger EDGE model 3001 and Barologger EDGE model 3001, Solinst Canada LTD). The hydraulic conductivity was estimated using a mathematic model combining the mass conservation principle and Darcys Law, as described by Pedescoll *et al.* (2009):

$$k = \frac{d^2 ln\left(\frac{2L}{d}\right)}{8Lt} ln\left(\frac{h_1}{h_2}\right) \qquad \textbf{(6-1)}$$

Where k is the hydraulic conductivity (m/s), d is the internal diameter of the steel pipe (m), L is the length of submerged pipe (perforated region) (m), t is the drainage time (seconds), h_1 is the water height at time zero and h_2 is the water height at time t.

6.2.5 Sampling and Analysis

Sampling campaigns were conducted on a weekly basis, with 1L grab inlet and outlet samples collected in plastic sampling bottles which were transported immediately to the laboratory for same day analysis. The grab samples were analysed for chemical oxygen demand (COD), ammonium-nitrogen (NH_4-N), nitrite-nitrogen (NO_2-N), nitrate-nitrogen (NO_3-N), total nitrogen (TN) and total phosphorus (TP) using colourmetric test kits as described in the manufacturers protocol (Hach, Germany). Total and volatile suspended solids (TSS/VSS) were analysed as per the standard methods (APHA, 2005), using glass microfiber filters with a particle retention size of 1.2µm. The pH, Dissolved Oxygen (DO) and Oxidation-Reduction (Redox) Potential (ORP) of the influent and effluent grab samples was determined at the laboratory using probes and a multi-meter (LDO sensor, pH gel electrode and ORP electrode; Hach, Germany). Additionally onsite analysis was conducted on water samples collected within the internal sampling pipes to determine pH, DO and ORP using rugged probes (LDO sensor, pH gel electrode and ORP electrode; Hach, Germany) and a portable multi-meter (HQ40d multi-meter; Hach, Germany).

6.2.6 Statistical Analysis

Statistical analysis to determine the differences between TSS, COD and TP load removals within the two groups of VFWs with different resting periods (DF4: VF1, VF2, VF3 and VF4; and DF45: VF5, VF6, VF7 and VF8), was conducted using one-way ANOVA, using a confidence level of $\alpha = 0.05$. Statistical differences of TSS, COD and TP load removals between VFWs with different dosing frequencies (VF1 & VF5; VF2 & VF6; VF3 & VF7; and VF4 & VF8) was determined using two sample t-tests. Analyses were conducted using Minitab statistical software (Version 17; Minitab inc, Pennsylvania State University, USA).

6.3 Results and Discussion

6.3.1 Hydraulic Behaviour

No hydraulic overload events were observed throughout the trial across the eight wetlands even though the highest HLR tested was 0.8m/d. This extends the maximum operable HLR during the course of trials with these pilot plants, from the maximum operable HLR of 0.19m/d during the first trial with unplanted beds (Chapter 4) and 0.4m/d with planted beds testing different dosing frequencies during the second set of trials (Chapter 5). In total, 20 months of operation had occurred prior to the current trials and this supports the suggestion that optimum operation requires stabilisation of the beds which takes between 1-3 years (Chazarenc and Merlin 2005; Gomez, 2016). Importantly, the operable HLR during the different trials all exceeded the typical HLR use for single stage VFW treating secondary wastewater of 0.05m/d (Brix and Arias, 2005) with the current trials exceeding the HLR used when rotating beds of 0.37m/d even though the beds where continuously used. Previous trials on VFWs for tertiary treatment have operated at HLRs of 0.27m/d in parallel (Schönerklee et al., 1997) and 0.45m/d on the operating VFW for four beds operating rotationally (Cooper et al., 1997). The current work suggest even higher HLR are possible for tertiary VFWs, and therefore operational practice from VFWs used on upstream applications may not be necessary in tertiary application, due to differences in the influent characteristics and concentrations.

The hydraulic conductivity (HC) within all the VFWs varied during the trial with a general decrease between the start and the end of the trial (Table 6-2). To illustrate, in the case of VF4 (HLR = 0.8m/d, DF = 4, no rest period) the monthly HC were 2.34, 1.34, 2.46, 2.31, 1.67 and 1.79m/d for June, July, August, September, October and November, respectively (Table 6-2). Higher HC were reported during the summer months but the difference in HC could not be solely accounted for by temperature with, for instance, temperature normalised (through an Arrhenius correction to 20°C) HC values for August and November of 2.77m/d and 2.48m/d respectively (VF2). The elevated HC in summer, when the temperature was warmer, is congruent with increased biofilm activity and hence better management of the applied load. No direct trend was observable in relation to the weekly averaged HLR, dosing frequency or rest schedule indicating that within the ranges tested the operational factors did not particularly influence the hydraulic character of the beds. For instance, increasing the number of rest days at a dosing frequency of 4 from 0 to 1, 2 and 3 resulted in HCs of 2.46, 1.75, 2.37, 2.61m/d in August and 1.67,1.39, 2.09 and 1.5m/d in October, respectively.

Table 6-2 Monthly average hydraulic conductivities from duplicated results for each VFW.

	Hydraulic Conductivity (m/day)							
	VF1	VF2	VF3	VF4	VF5	VF6	VF7	VF8
DF	4	4	4	4	45	45	45	45
HLR	0.46	0.57	0.69	0.80	0.46	0.57	0.69	0.80
Feed:Rest	4:3	5:2	6:1	7:0	4:3	5:2	6:1	7:0
SLR*	9.2	11.4	13.7	15.9	9.2	11.4	13.7	15.9
June	0.86	2.45	1.89	2.34	2.98	2.18	2.14	1.46
July	1.81	*8.71*	1.11	1.34	1.80	0.98	*17.63*	1.02
August	2.61	2.37	1.75	2.46	1.75	2.52	4.08	2.14
September	2.32	2.38	1.39	2.31	1.89	1.44	1.63	2.48
October	1.50	2.09	1.39	1.67	1.77	0.85	1.85	1.48
November	1.62	1.28	0.99	1.79	2.21	1.03	1.35	1.22

*Median solids loading rate based on influent suspended solids (gTSS/m^2/d).

Further analysis of the hydraulic performance was undertaken by establishing the monthly average solids, organic and nutrient loadings and removals. As HC measurements were conducted monthly, the pollutant loadings and removals were averaged over each month to calculate the correlations, and adjusted to represent the effective loading rather than the loadings solely obtained on the feeding day. No overall correlations were apparent in regards to TSS loading, COD loading or ammonia loading (Figure 6-2) implying that the systems could be operated at higher rates than occurred in this trial. Importantly, the HC remained stable across the whole range of solids loading rates tested (Figure 6-2A). For instance, the HC was 2.2m/d at the lowest solids loading rate of 6.8gTSS/m^2/d and 1.2m/d at the highest solids loading rate of 20gTSS/m^2/d. These levels exceed the loading rates reported to avoid clogging in secondary single stage VFW of 5gTSS/m^2/d (Winter and Goetz, 2003) but are below the 45gTSS/m^2/d that has been reported to result in permanent clogging of the media (Langergraber et al. 2003). The highest solids loading rates were associated with the no rest operation and this did not impact on the HC, implying that at the maximum loaded rate tested there was no requirement to rest the beds to provide greater opportunity for the biofilm to manage the applied load. Similar trends were observed between COD loading and HC (Figure 6-2B) and ammonium-nitrogen loading and HC (Figure 6-2C). In the case of COD loading this extended the previously reported guide level of 20gCOD/m^2/d up to at least 42.5gCOD/m^2/d. No significant difference was observed in relation to dosing frequency, which contradicts established thinking for VFW used for raw and secondary treatment (Bancolé *et al.,* 2003). In those cases the higher dosing frequencies are reported to focus the activity of the bed within the upper most 30cm leading to a greater risk of clogging. The current results support the previous trial that showed the dosing frequency is not a major influence on the hydraulic efficiency of tertiary VFWs (Chapter 5). It is posited that the low contaminant level in the water alters the loading profile and desensitise the operation of the bed with respect to dosing frequency. However, if higher loads were to be applied it may be that dosing frequency becomes important and as such should remain an active area of consideration.

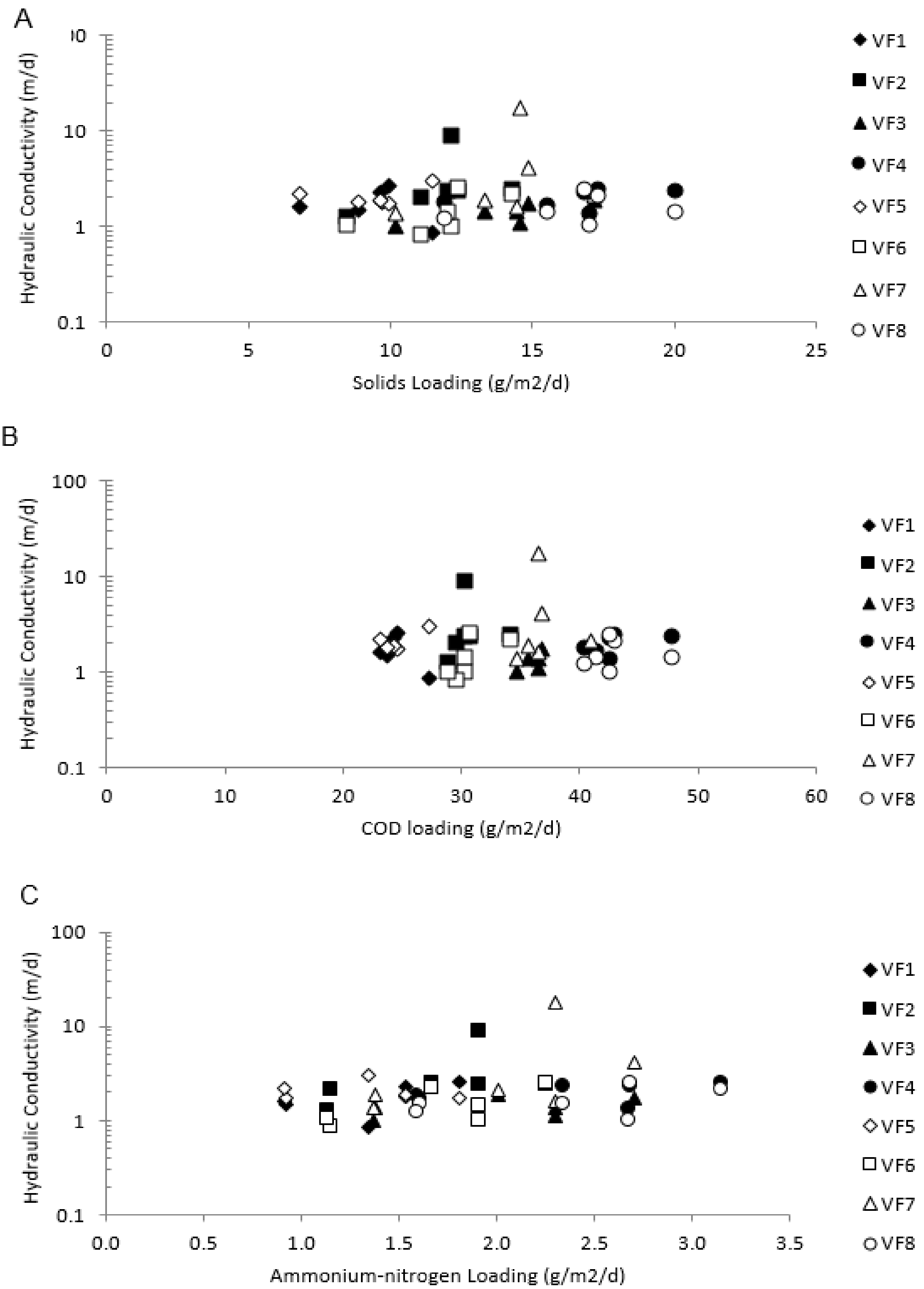


Figure 6-2 Relationships between Hydraulic conductivity and pollutant loading (A) Total suspended solids, (B) Chemical Oxygen Demand and (C) Ammonium-nitrogen loading.

6.3.2 Solids and Organics Removal

The VFW were observed to be effective at removal of suspended solids irrespective of the specific operating conditions chosen (Figure 6-3A). Residual suspended solids varied between 3.4±1.2mgTSS/L and 5.6±4.7mgTSS/L representing removal efficiencies of 72% and 84%. Across the trial, statistical analysis (one-way ANOVA) confirmed there were no significant differences in TSS load removals between VFWs operating within the same dosing frequency group (DF4/DF45) but with different resting periods (VF1 – VF4, DF4: P=0.661; VF5 – VF8, DF45: P=0.125). Additionally, a two sample t-test confirmed there were no statistically significant differences determined between VFWs receiving the same rest periods, and different DF (VF1 & VF5, 3 days rest: p=0.162; VF2 & VF6, 2 days rest: 0.226; VF3 & VF7, 1 day rest: 0.878; and VF4 & VF8, no rest: p=0.089). Greater variation was seen at higher dosing frequencies with one bed, VF4, exceeding 10mgTSS/L over a four week period. Thereafter the bed returned to similar performance as the others remaining below 5mgTSS/L for the remaining seven weeks of the trials (Figure 6-3C).

A similar performance was observed with respect to Chemical oxygen demand (COD) which decreased from an average influent value of 53mgCOD/L to less than 30mgCOD/L across the entire trial for all the wetlands (Figure 6-3D). VFW operating with a higher dosing frequency produced slightly lower residuals with an average range of between 14.4mgCOD/L and 20.6mgCOD/L for DF45, compared to average effluent concentrations of between 17.3mgCOD/L and 21.4mgCOD/L for DF4 (Figure 6-3B). Statistical analysis, conducted using a two sample t-test, showed there was a significant difference between the COD load removed in VF4 compared to VF8 (P=0.0189), both receiving no resting periods and different dosing frequencies. This corresponded to removal efficiencies ranging between 51-74% for VF4 and 59-80% for VF8. In addition, one-way ANOVA confirmed significant differences between the COD load removals of VF5, VF6, VF7 and VF8, all receiving DF45 (P=0.0012).

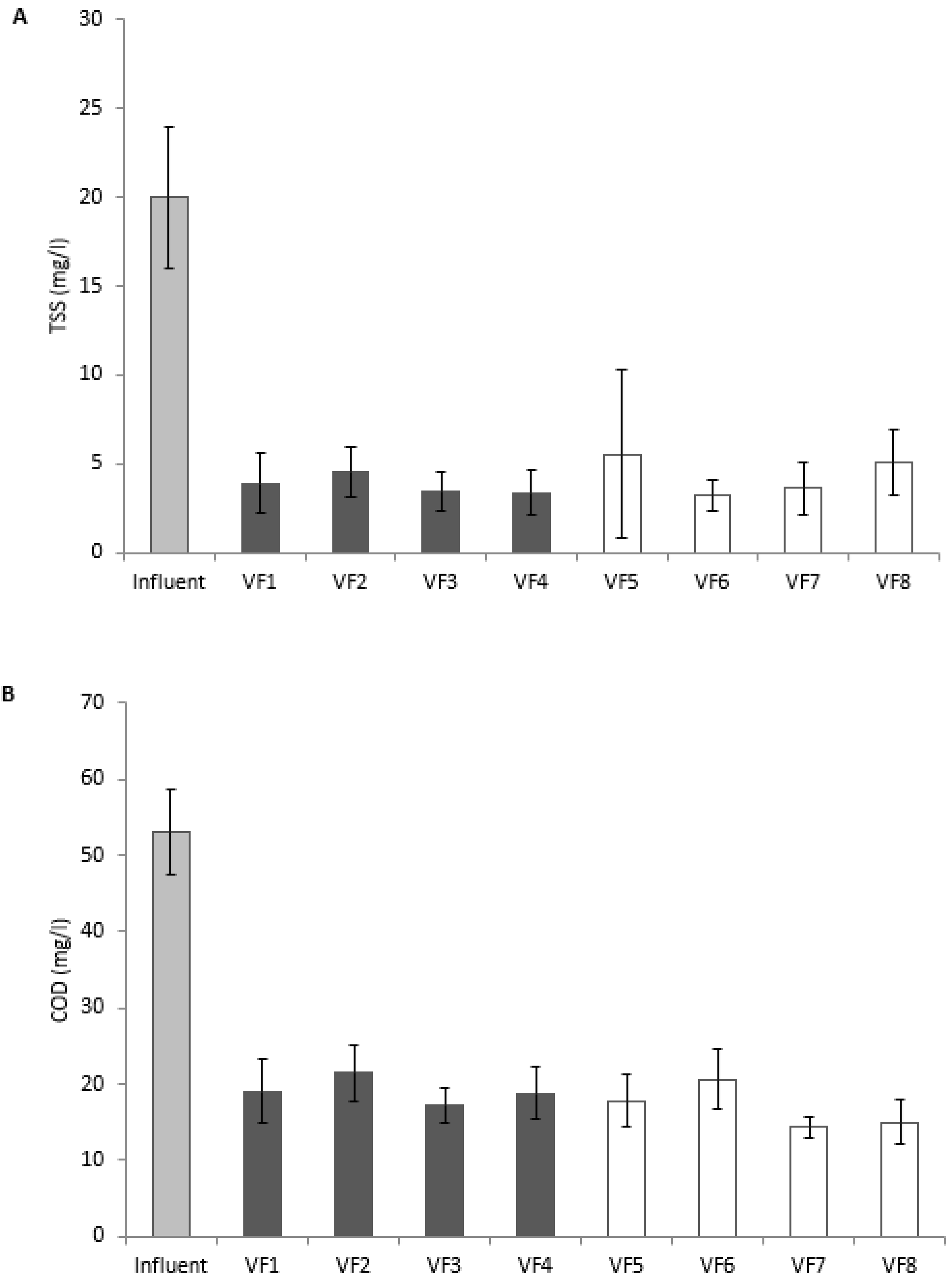
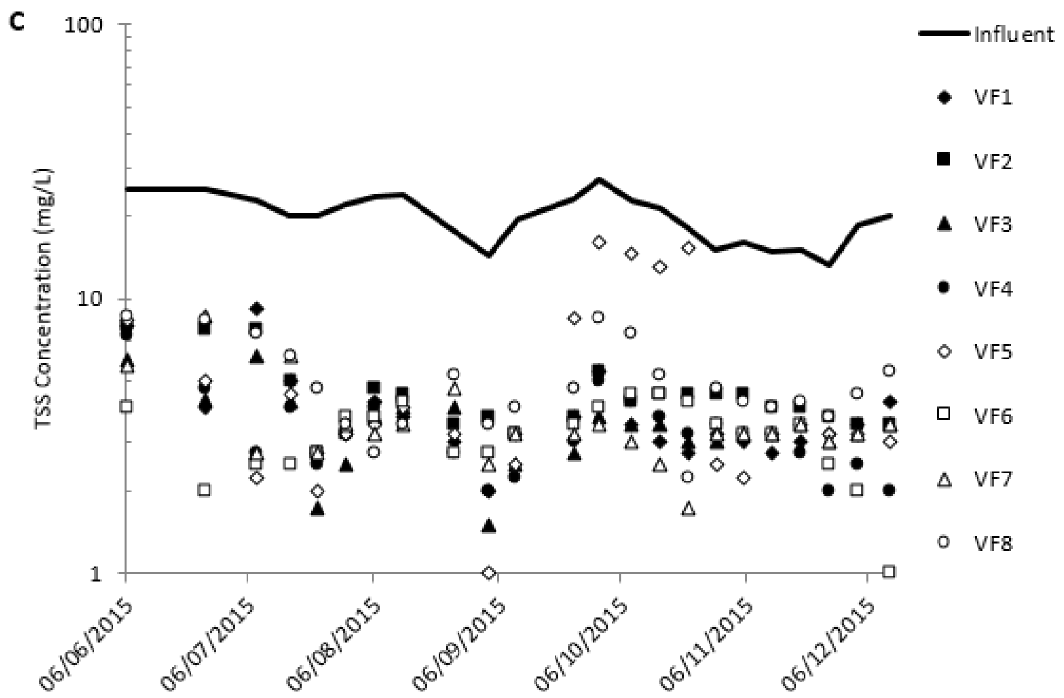


Figure 6-3 (A) Influent and effluent TSS concentrations with standard deviation. (B) Influent and effluent COD concentrations with standard deviation. The light grey bars show the influent, the dark grey are the DF4 VFW group and the white are the DF45 VFW group.



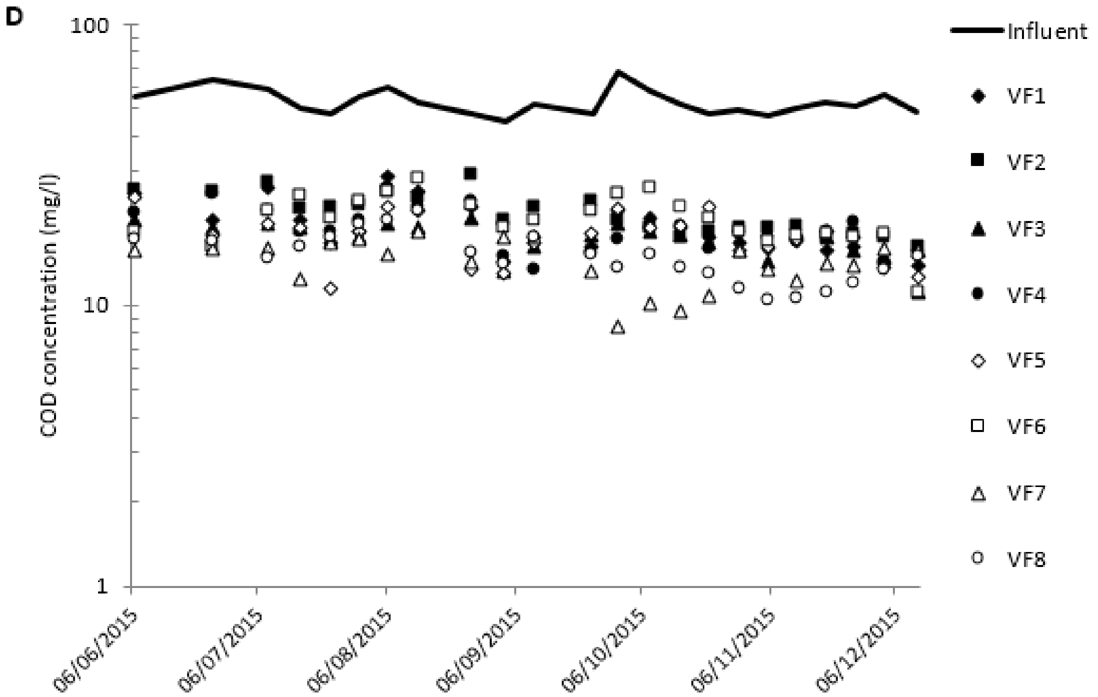


Figure 6-3 (C) Timeline of TSS influent and effluent concentrations. (D) Timeline of COD influent and effluent concentrations.

6.3.3 Nutrient Removal

The overall data shows almost complete removal of ammonium-nitrogen in seven of the eight VFWs producing average effluent concentrations of between 0.005mgNH_4-N/L and 0.02mgNH_4-N/L, equating to removal efficiencies of ≥99.3% (Figure 6-4A). The exception was VF6, which produced an average effluent ammonium-nitrogen concentration of 0.19±0.099mgNH_4-N/L (93.2% efficiency) demonstrating a sustained level of treatment across the entire trial period and evidencing that tertiary vertical flow wetlands can generate very low effluent ammonia concentrations even when operated at elevated hydraulic loading rates. Although removal of ammonium-nitrogen was substantial across all the VFWs, a slightly lower removal was observed within the higher dosing frequency group (DF45) although the effluent concentration did not exceed 0.1mgNH_4-N/L apart from VF6, which remained below 1mgNH_4-N/L throughout. This bed revealed a slight dissolved oxygen decrease of 0.29mgO_2/L whilst all other beds consistently showed a DO increase of up to 1.8mgO_2/L. The reduction in VF6 did not reduce the total DO below the nitrification threshold and hence still nitrified, but may be indicative of a period of partial clogging that reduced oxygen transfer. This is supported by the bed operating with the lowest hydraulic conductivities of all the beds during the trial (Table 6-2). All VFWs demonstrated an increase in nitrate of between 1.3mgNO_3-N/L and 2.2mgNO_3-N/L and a reduction in nitrite. The total nitrogen deficit is attributed to the production of nitrogen gas or nitrous oxide during incomplete nitrification/denitrification reactions, or through other removal mechanisms such as plant uptake (Maltais-landry et al., 2009).

Overall total phosphorus was removed consistently across all VFW for the entire trial period (Figure 6-4D). The average concentration decreased from an influent level of 1.1±1.8mgTP/L down to between 0.7 and 0.22mgTP/L. One-way ANOVA statistical analysis confirmed a significant difference in load removals between VFWs in the higher dosing frequency group (DF45: VF5, VF6, VF7, VF8) (p=0.0113). Comparing the data between corresponding VFWs within both the high and low dosing frequency groups, two sample t-tests confirmed a significant difference in TP load removals between VF3 and VF7 (1 day rest, DF4 and DF45:

p=0.0340), and between VF4 and VF8 (no rest, DF4 and DF45: p=0.0023), with the higher dosing frequency VFWs performing significantly better (removal efficiencies: VF3: 62.0%; VF4: 59.8%; VF7: 70.3%; and VF8: 71.8%). The site removes phosphorus through chemical dosing with iron coagulants and as such the majority of the phosphorus is in the particulate and colloidal form. Previous assessment has indicated that approximately 80% of the total phosphorus is in the particulate form and 6% residing in colloid sizes. Accordingly, the major removal pathway is solids capture rather than adsorption and, as such, the ability to remove phosphorus will not be limited by the capacity of the media directly.

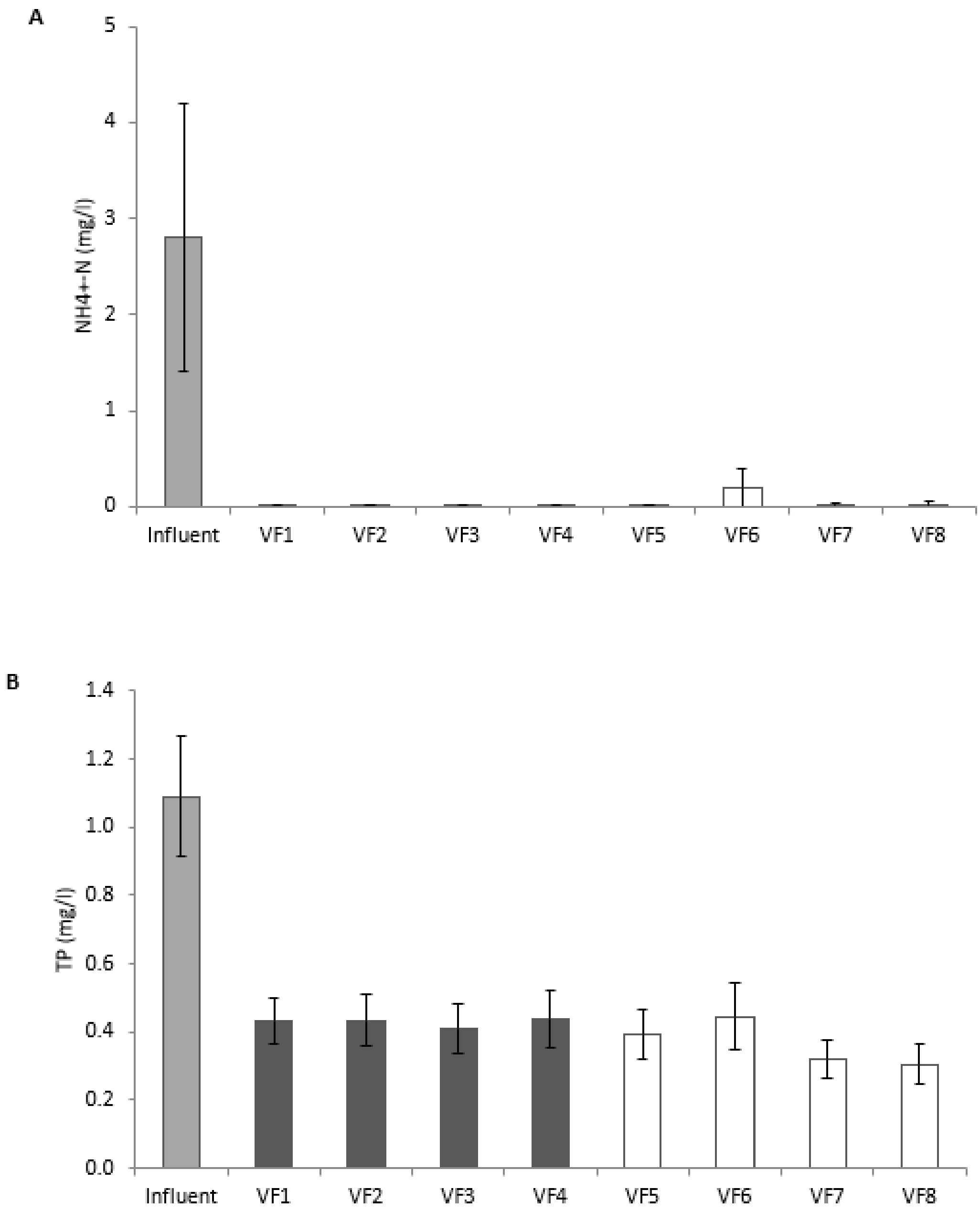


Figure 6-4 (A) Influent and effluent NH4-N concentrations with standard deviation. (B) Influent and effluent TP concentrations with standard deviation. The light grey bars show the influent, the dark grey are the DF4 VFW group and the white are the DF45 VFW group.

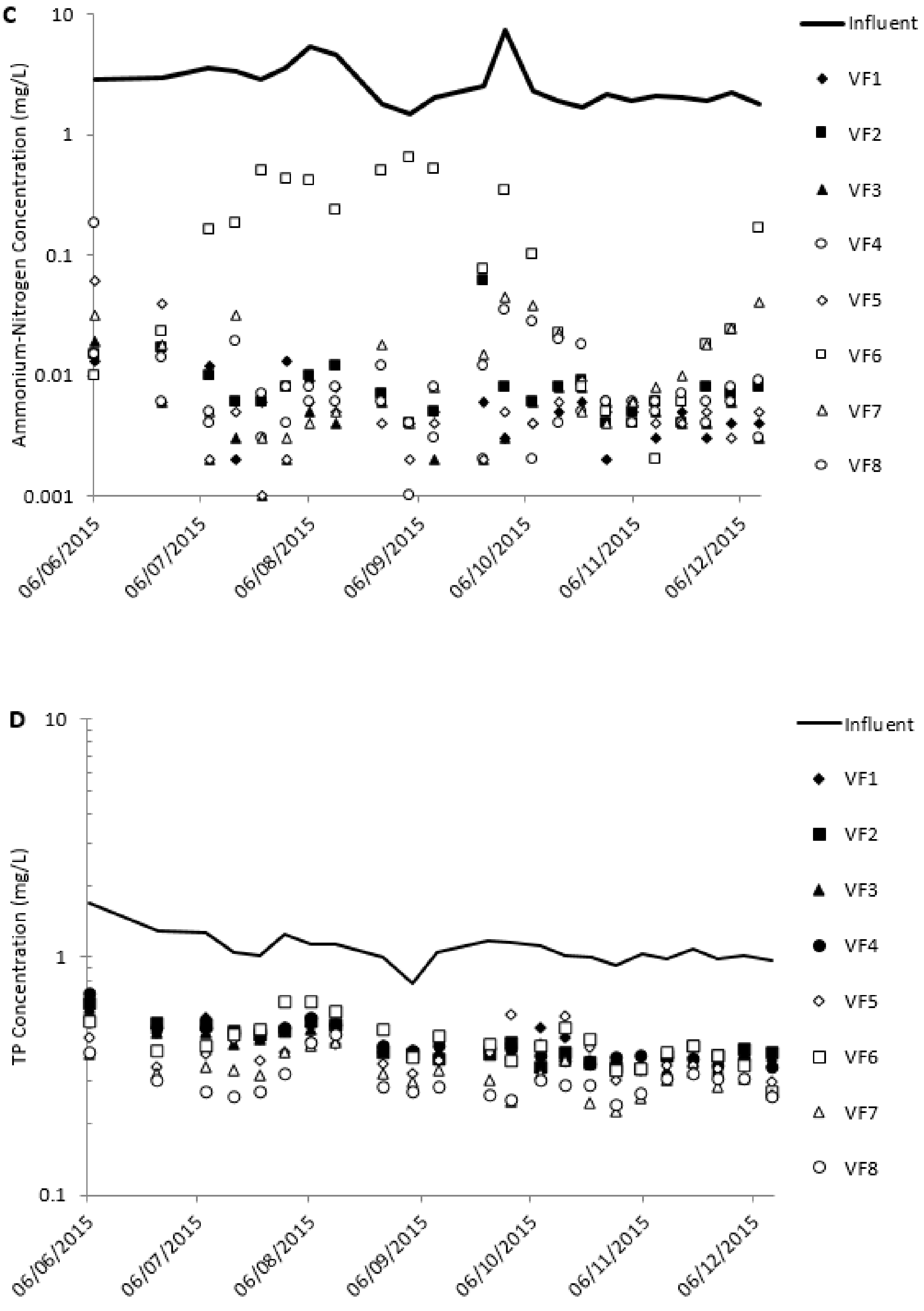


Figure 6-4 (C) Timeline of NH_4-N influent and effluent concentrations. (D) Timeline of TP influent and effluent concentrations.

6.4 Conclusions

Vertical flow wetlands are demonstrated to be effective as a tertiary treatment process for use on sites that require low ammonia discharges. Overall a sub 30, 10, 1 ppm COD, suspended solids and ammonia is achievable offering a potential to meet tight discharge standards whilst retaining the near passive nature of constructed wetlands. The performance was not significantly influenced by either the instantaneous hydraulic loading rate, rest period or dosing frequency suggesting that the operating limits have yet to be reached. Application of vertical flow wetlands for tertiary treatment can also operate at hydraulic loading rates of at least 0.8m/d which exceeds those reported for other applications of vertical flow wetlands and offer promise for even more compact systems in the future as the limiting hydraulic loading rate has yet to be reached.

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Chapter 7

Tertiary vertical flow constructed wetlands: Understanding the impact of design choices on the potential economic viability in meeting tight ammonia discharge standards on small works

7 Tertiary Vertical Flow Constructed Wetlands: Understanding the Impact of Design Choices on the Potential Economic Viability in Meeting Tight Ammonia Discharge Standards on Small Works

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Abstract

It is anticipated that tightening nutrient consents will be applied to wastewater treatment works discharges within the coming years, with the intention of improving the water quality of surface waters (Water Framework Directive). In terms of ammonia, this could result in effluent discharge consents of ≤1mgNH_4-N/L. Whilst conventional tertiary treatments have shown potential to perform to within these proposed consents, they often require additional energy use in the form of continual pump or air blower operation to achieve desired flows and residual dissolved oxygen levels. Vertical flow wetlands offer a low energy alternative, whilst providing comparable, if not better, treatment efficiency. Vertical flow wetlands have the potential to operate as energy neutral systems, dependant on the site topography. Alternatively, if the use of siphons is not feasible, pump associated energy costs remain low due to the intermittent feeding regime applied. This paper undertakes an economic assessment on VFWs designed for population equivalents of 100, 500 and 2000, to identify key challenges in implementation through CAPEX, OPEX and Whole Life Cost estimates. To determine the economic feasibility of the VFW as a tertiary nitrifying treatment, an economic comparison was conducted against nitrifying submerged aeration filters (NSAF) and aerated horizontal flow wetlands (AHFW) designed to

the same population equivalents. Initial findings show VFWs to be an economically viable technology for tertiary application, producing overall CAPEX, OPEX and whole life costs similar to that of the AHFWs and below that of NSAF.

7.1 Introduction

Small wastewater treatment works are defined as having a population equivalent (pe) of 2000 or less, and account for over 75% of all wastewater treatment works in the UK (Upton, 1995). As a relatively small flow passes through these sites, they have traditionally had no ammonia discharge consents when discharging to a stable receiving watercourse and an ammonia discharge consent of 5mgNH_4-N/L when discharged to a sensitive watercourse or to a site of special scientific interest (Johnson *et al.*, 2007). It is anticipated that with invested interest from regulatory bodies (Water Framework Directive 2000/60/EC; US EPA, Clean Water Act Action Plan, 2009), more stringent nutrient discharge consents will be applied to small works in the coming years, with a likely trend towards ammonia effluent concentrations of ≤1mg/L to bring into line with existing river water quality, and potentially as low as ≤0.6mg/L for sensitive watercourses or areas of special scientific interest.

Historically, tertiary treatments, such as horizontal flow wetlands (HFWs), have been applied to small wastewater treatment works to provide the required polishing total suspended solids (TSS) and biochemical oxygen demand (BOD) discharge consents. In anticipation of the proposed stringent ammonia consents, water companies are investing in projects to define ways in which current secondary wastewater treatment sites can be upgraded to incorporate an ammonia removal polishing asset or how existing tertiary sites can be modified to enhance nitrification. Traditionally, delivering additional nitrification capacity in tertiary treatment was accomplished through the use of tertiary aerobic biofilm processes such as nitrifying submerged aerated filters (NSAF). These are relatively capital intensive and exert a relatively large energy demand compared to the rest of the treatment works. In more recent times, adaptation of horizontal flow wetlands through the inclusion of artificial aeration (AHFW) has gained acceptance as it is delivered through relatively simple alteration. Although these technologies are able to achieve ammonia effluent concentrations of 0.01mgNH_4-N/L and 1mgNH_4-N/L, for AHFW and NSAF, respectively (chapter 5; Butterworth et al., 2013), both require use of electrical equipment to maintain a high treatment efficiency which may prove to be problematic on small remote sites.

Vertical flow wetlands (VFWs) offer an ideal alternative to existing tertiary nitrification treatments, as they are able to perform to a high removal efficiency (>99%) to achieve effluent ammonia concentrations of less than 0.02mgNH_4-N/L (Chapter 6). In addition, VFWs are able to achieve high levels of solids and organic matter removal, and have shown potential in achieving total phosphorus (TP), heavy metals (HM) and pharmaceuticals and personal care products (PPCP's) (Stefanakis et al. 2014). In sites with available hydraulic head, it is possible to provide influent to the VFWs with the use of a siphon, rendering the technology as energy neutral. Therefore, the use of VFWs offer the potential to meet challenging ammonia discharge consents whilst retaining the passive attributes of wetland technology.

The current use of vertical flow wetlands for tertiary application is limited, primarily due to historical associations with poor treatment potential and the unacceptability of continual high hydraulic loadings, likely to have been mediated through lack of design and operational understanding. Recent research has been conducted to optimise operational strategies of a tertiary VFW, through alterations of the hydraulic loading rate (HLR), dosing frequency (DF) and resting periods (RP) (Chapters 4, 5 and 6). This paper applies these previous findings to the design of a VFW for tertiary nitrification to ascertain if there is the potential for VFWs to provide an economically appropriate alternative for tertiary treatment and hence is worthy of further development. To assess the economic viability of VFWs as a nitrifying tertiary treatment, capital expenditure (CAPEX), operational expenditure (OPEX) and whole life cost (WLC) comparisons have been conducted against NSAF and AHFW for a range of population equivalents, to establish potential critical cost components and limiting design values.

7.2 Business Case Scenarios

This study considers the upgrading of three small, secondary wastewater treatment works, with population equivalents (PE) of 100, 500 and 2000, to include a tertiary treatment for the intention of meeting tightening future ammonia discharge consents of 1mgNH_4-N/L. The upfront works includes primary settlement, biological secondary treatment and secondary settlement such that the current effluent contains 5mgNH_4-N/L. Three tertiary treatment options are considered: (A) vertical flow wetlands (VFW), (B) Aerated horizontal flow wetland (AHFW), (C) Nitrifying submerged aerated filter (NSAF). In all cases, it is assumed there is no existing tertiary treatment but storm water treatment exists for flows above 3 Dry Weather Flow (DWF). As the comparative tertiary technologies are established as successful nitrifying tertiary treatment processes, it is assumed that the design criteria incorporate sufficient ammonia removal to achieve within the proposed future discharge consent of 1mgNH_4-N/L. Design criteria used in this paper are based on the assumption of a water use equivalent of 180L/PE/day (Gray, 2008) and it is assumed that the water treatment works for upgrading has sufficient land space to accommodate the introduction of the tertiary treatment processes, and that purchase of additional land is not required. Further, it is assumed that the existing infrastructure is able to accommodate the additional technology in terms of power such that all associated costs with energy provision have been excluded.

7.3 Materials and Methods

7.3.1 Basic Design Parameters

The design of the VFW for this study was primarily based on findings from within the current thesis, which include design influences from within existing literature. The design specifications of the comparative tertiary treatments were obtained through a sponsoring water company and through existing literature. Design assumptions were used where appropriate.

7.3.1.1 Vertical Flow Wetlands

Typically, VFWs are constructed at ground level, requiring ground excavation work and inclusion of an impermeable geo-textile liner to prevent leaching of untreated waters into ground water sources. Vertical flow wetlands are designed to a total depth of 1m, inclusive of three media layers for wastewater treatment and a freeboard at the surface to accommodate incidence of hydraulic overloads during routine feeding (short term) or prolonged clogging events (long term). The influent to the wetlands is delivered through a network of distribution pipes above the VFW surface. Similarly, treated wastewater is collected through a pipe network that is stationed within the drainage media layer at the bottom of the wetland. Passive aeration is incorporated in the drainage system, which also acts as an overflow.

Based on the data in the current thesis, an established VFW can successfully operate under full scale tertiary application using a hydraulic loading rate of 0.8m/d at 3 DWF, with no prolonged resting periods, and planted with *Phragmites austalis* at a density of 4 plants/m^2 (Table 7-1). Data within the current thesis demonstrated that VFW performance and operation was not significantly impacted by dosing frequency (Chapter 5). An initial assessment of the cost implications indicated that the capital cost of the feeding system decreased with increasing dosing frequency and became relatively stable once the dosing frequency exceeded 12doses/day and so was set nominally at 24 feeds/day. Influent wastewater may be delivered to these systems through use of pumps or dosing siphons depending on the head loss and layout of the site. For the purpose of this study a cost comparison between pumping and siphoning options have been conducted and include the design of a submerged concrete sump designed to a size of 1.5 times the volume of one batch feed. The VFWs for use in this study were designed based on at least two side by side beds operating in parallel with the maximum size of an individual bed set at 500PE to ensure even distribution across the bed during feeding consistent with current Austrian design guidelines.

Table 7-1 Summary of main design parameters of a VFW system

	Value			
Main Design Parameter	**100pe**	**500pe**	**2000pe**	**Unit**
In flow	54	270	1080	m^3/d
Design footprint	0.57	0.57	0.57	m^2/pe
Number of beds	2	2	4	no of beds
Total footprint	57	285	1140	m^2
Total depth	1	1	1	m
Main treatment media: sand diameter	≤4	≤4	≤4	mm
Main treatment media: sand depth	0.5	0.5	0.5	m
Transition media: gravel diameter	4-8	4-8	4-8	mm
Transition media: gravel depth	0.1	0.1	0.1	m
Drainage media: gravel diameter	16-32	16-32	16-32	mm
Drainage media: gravel depth	0.15	0.15	0.15	m
Freeboard Height	0.25	0.25	0.25	m
Phragmites australis	228	1140	4560	no. of plants
Feed pump	0.12	0.62	1.23	kW

7.3.1.2 Aerated Horizontal Flow Wetlands

As with VFW, design criteria recommend full scale AHFW are installed at ground level, which will require excavation and lining with an impermeable geo-textile liner. Aerated horizontal flow wetlands are designed with an overall depth of 1.3m including a media depth of 0.6m, filled with gravel of a diameter of between 6-12mm (Table 7-2) with a freeboard height of 0.6m to accommodate potential hydraulic overloads. Influent is delivered to the wetland through an inlet trough at one end of the wetland and is dissipated over gabions of a diameter of between 50-200mm. The base of the wetland is set on a 1% decline slope towards a collection pipe at one end of the wetland. In aerated systems, air lines with a 12mm diameter are positioned along the bottom of the bed and release air through 2mm diameter perforations routinely spaced at 300mm lengths (Butterworth et al., 2013). A recommendation of 0.7m^2/pe is required for sufficient treatment at tertiary application (Butterworth, 2014; Cooper, 1993). The air is provided to these systems by continually operated blowers. As AHFW are not dependant on batch feed operation, they can be gravity fed, thus eliminating the need for influent pumping. As with VFW, aerated horizontal flow wetlands are planted with *phragmites australis* at a density of 4 plants/m^2.

Table 7-2 Summary of main design parameters of an AHFW system

	Value				
Main Design Parameter	**100pe**	**500pe**	**2000pe**	**Unit**	**Reference**
Flow	54	270	1080	m^3/d	
Design footprint	0.7	0.7	0.7	m^2/pe	(Cooper, 1993)
Total footprint	70	350	1400	m^2	
Blower	0.24	1.2	4.83	kW	(Butterworth, 2014)

7.3.1.3 Nitrifying Submerged Aeration Filter

Nitrifying submerged aerated filters (NSAF) are constructed above ground using either metal, concrete or glass reinforced plastic (GRP) tanks. These structures contain either fixed media or randomly packed media that are generally constructed from plastic. Influent to the NSAF can either enter through the top or the bottom of the system. A continuous air supply is provided at the base of the system with the use of a blower and air compressor. Backwashing of the media is performed for 30 minutes every day to reduce solids build-up in the media and to prevent long term blockages. Nitrifying submerged aerated filters have a smaller footprint when compared to that of wetlands (Figure 7-1), and are designed based on an ammonia influent loading of between 0.01 and 0.4kgNH_4-N/m^3 of media/d. A value of 0.04kgNH_4-N/m^3 was selected to reflect the tight effluent discharge and post discussion with water company personnel (Vale, 2017) (Table 7-3).

Table 7-3 Summary of main design parameters of a NSAF system

	Value				
Main Design Parameter	**100pe**	**500pe**	**2000pe**	**Unit**	**Reference**
Flow	54	270	1080	m^3/d	
Design footprint	0.015	0.015	0.015	m^2/pe	
Total footprint	1.5	7.5	30	m^2	
Energy demand: Feed pump	0.12	0.62	1.23	kW	
Energy demand: Air blower	0.24	1.2	4.83	kW	(Butterworth, 2014)
Energy demand: Backwash pump	0.12	0.12	0.12	kW	

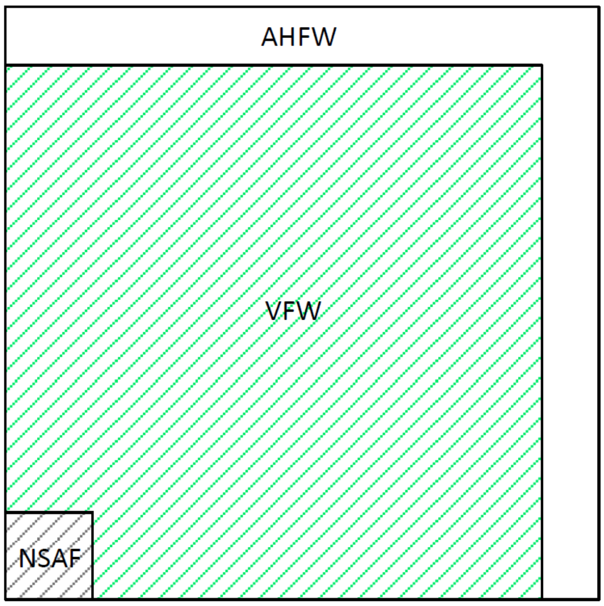

Figure 7-1 A relative total footprint comparison between AHFW, VFW and NSAF. The sizing was based on area per person (m^2/pe) for each technology (AHFW: 0.7m^2/pe; VFW: 0.57m^2/pe; and NSAF: 0.015m^2/pe). The dimensions were determined using the square root of the area (AFHW: 0.837m x 0.837m; VFW: 0.755m x 0.755m; and NSAF: 0.122m x 0.122m). The image was drawn to scale using Microsoft Visio Professional 2016.

7.4 Economic Evaluation

7.4.1 Capital Cost Estimates

The capital expenditure (CAPEX) for the VFWs in this study were estimated based on scale up calculations using costs incurred for the major design components during pilot plant construction (Appendix 1). The estimated CAPEX costs for both the AHFW and NSAF were obtained through water company data. The AHFW was based on the capital cost of a non aerated HFW and the cost of the aeration system excluded to provide a conservatively low estimate for comparison. To ensure a consistent basis for comparison, the cost of the HFW was estimated using the design approach utilised for costing the VFW, taking into consideration the cost of excavation, land preparation, liner, media and plants and assuming a baseline pricing structure (ie: all gravel sizes having the same costs), but adjusted to HFW design numbers. This estimated cost was compared to the provided water company cost data and the ratio of the two used to adjust the cost estimates of the VFW. The adjustment factors were determined to be 16.14, 8.72 and 5.65 for the 100pe, 500pe and 2000pe sites, respectively. Pump costing data were obtained from pump cost curves within the literature (Loh et al., 2002). The stated ancillary costs for this study include inlet, outlet and internal pipe work for the VFWs, duty and standby feed pumps, a concrete sump with a height of 2m designed to hold 1.5 times one batch feed and all associated fittings and controls. Excavation costs for the study were estimated for an excavation of 1.5m. The CAPEX for all three technologies included estimated costs for upgrading sites with population equivalents of 100, 500 and 2000 (Table 7-4). Uncertainty in the cost estimates is reflected in a ±30% error band to assess feasibility and provide initial comparisons between the three different technologies.

7.4.2 Operational Cost Estimates

The operational expenditure (OPEX) for the three technologies were estimated based on data obtained from literature and from the sponsoring water companies. The operational costs include energy requirements and usage, manual labour for onsite maintenance and refurbishment requirements (Appendix 2-4). The hourly rate of pay for an onsite operator conducting asset maintenance was provided by a sponsoring water company at a rate of £26.50/hour, the electrical costs for an industrial user was priced at £0.085/kWh (Whitton, R, 2016) and wetland refurbish was estimated at an occurrence of once every 10 years (Knowles et al., 2011) with a cost of £30/m^2 (UKWIR, 2011). Energy demand for the AHFW and NSAF air blowers were determined on the assumption of an energy requirement of 58Wh/pe/d (Butterworth, 2014) and the backwash pump was assumed to be rated at the same power as the feed pump. Operation expenditure costs incurred for the NSAF did not include asset refurbishment.

7.4.3 Whole Life Costs

An asset lifespan of 40 years (water company recommendation) is considered for whole life costing (WLC), which is determined from both the initial CAPEX and annual OPEX including energy, maintenance and refurbishment expenditure and assumes a discount rate of approximately 7%. This is considered in the following simplified calculation used to determine the WLC, which was provided by a sponsoring water company:

$$WLC = CAPEX + (OPEX\ x\ 14) \tag{7-1}$$

Full whole life cost calculations were compared to the simplified calculation to assess its viability (Appendix 5).

Table 7-4 Summary of capital expenditure estimates for major design components of VFW (with both pump and siphon feeding), AHFW and NSAF for design capacities of 100, 500 and 2000 population equivalents. Note that costs do not include design or planning fees prior to construction.

		CAPEX (£)		Reference
VFW	**100 pe**	**500pe**	**2000pe**	
Excavation	298.4	1491.98	5967.9	(ICE, 2010)
Land preparation	11.97	59.85	239.4	(ICE, 2010)
liner	432.63	1598.85	6312.24	Pond liners: http://www.geosynthetic.co.uk/
Main treatment media	1067.04	5335.2	21340.8	Tarmac aggregates: http://www.tarmac.com/
Transition media	123.12	615.6	2462.4	
Drainage media	184.68	923.4	3693.6	
Plants	285	1425	5700	*Phragmites australis*: http://www.reedsfromseeds.co.uk/
BASE TOTAL	**2402.835**	**11449.88**	**45716.34**	
Adjusted cost	38781.76	99842.91	258297.3	Based on validation cost factor
Ancillaries (pumps)	7330	11246	15150	(Loh et al. 2002)
SCALED UP TOTAL	**46111.76**	**111088.9**	**273447.3**	
Ancillaries (siphons)	1234	1605	2942	*based on a siphon cost of £500/unit: assumption
SCALED UP TOTAL	**40015.76**	**101447.9**	**261239.3**	
AHFW (including all civils work)	**34,581**	**102,278**	**260,274**	(P. Vale, 2017, pers.comm)
NSAF				
Civil costs	7970	25350	68880	
M&E	97477	163935	256529	(P. Vale, 2017, pers.comm)
TOTAL	**105447**	**189285**	**325409**	

7.5 Results and Discussion

7.5.1 Capital Cost Estimates

The estimated capital cost of the different options revealed an order of costs as AHFW<VFW(s)<VFW<NSAF (Figure 7-2), where VFW(s) represents a siphoned VFW. To illustrate at a population equivalent of 100, the estimated CAPEX was £34,581; £40,015; £46,111; and £105,447 for the AHFW, VFW(s), VFW, and NSAF respectively. The difference between the different wetlands was relatively small and within 9% so can be considered economically comparable. The increased CAPEX of the VFW compared to the AHFW can be attributed to an increased cost associated with the media as the sand used in the VFW is 60% more expensive per unit mass than the gravel used in the HFW. Both the VFW and AHFW technologies have a significantly lower CAPEX compared to that of the NSAF. However, this becomes less evident with increasing population equivalent. The energy neutral, siphoned VFWs (VFW(s)) have a very similar CAPEX as the AHFW throughout each of the three size designs. The conservative wetland CAPEX costing estimated for this study is considerably higher than those suggested by Vymazal and Kropfelova, (2008) at £200/m^2, £100/m^2 and £50/m^2 for wetlands with sizes of 100m^2, 1000m^2 and 5000m^2, respectively (Table 7-4). The equivalent numbers in the current study are £346/PE, £205/PE and £130/PE for the HFW for the 100, 500 and 2000 PE sites compared to £460/PE, £222/PE and £137/PE for the VFW using the dosing siphon.

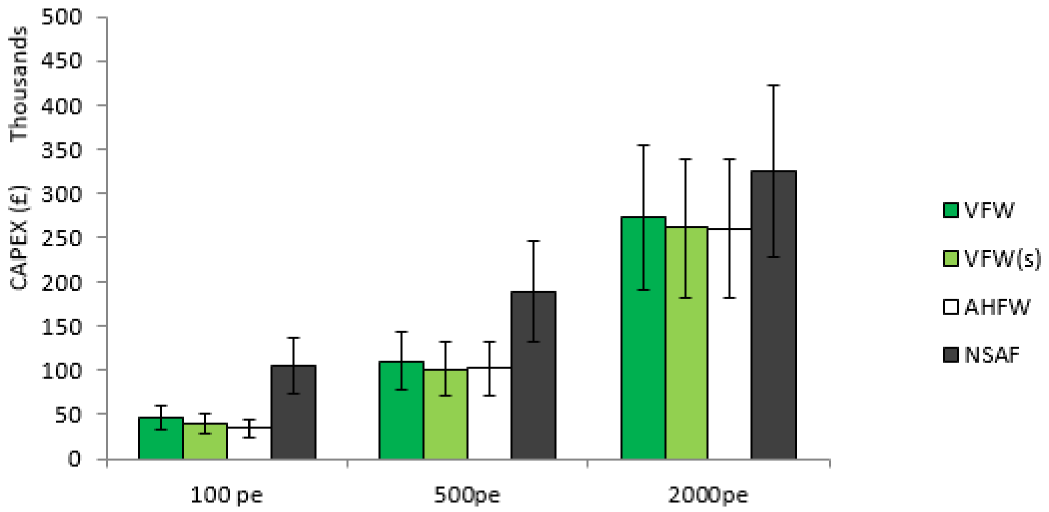

Figure 7-2 CAPEX estimations for VFWs with pumps and siphons VFW(s), AHFWs and NSAFs for design capacities of 100, 500 and 2000pe (± 30% variation).

Analysis of the capital cost of the VFWs identified the major initial cost critical components to be the media, collectively accounting for 48%, 54% and 57% of the CAPEX for 100pe, 500pe and 2000pe wetland designs, respectively (Figure 7-3). This is in agreement with USEPA, (1999) whereby media costs comprised 48% of the total CAPEX VFW costs. A consideration for reduction in the CAPEX costs of VFW systems would be to substitute sand media for gravel which, and based on cost data obtained during pilot plant construction, would bring down media costs down by 61.5%, equating to overall CAPEX reductions of 14%, 16% and 17% for 100pe, 500pe, and 2000pe designed VFWs, respectively.

The exact size of media that is required remains an area of further investigation and as such, the potential impacts of switching to coarser media cannot be confirmed. However, the current trials with sand demonstrated excellent effluent ammonia and stable DO profiles suggesting that the system was able to provide greater nitrification capacity than was actually required. Further, analysis of AHFW has shown that nitrification activities occur within the first half of the bed length within the gravel (Butterworth et al, 2014). Accordingly, the ability to reduce the capital cost of the VFW should be considered realistic. Another potential

CAPEX cost reduction is to use siphons as an alternative to pumping. However, this is highly dependent on the topography of the proposed site and the associated head losses. Cost comparisons are included within this report for both pumped and siphoned VFW designs and, predictably, show an overall slight reduction in cost within the siphoned VFW design.

Irrespective of the media chosen, the key variable associated to the capital cost is the size of the bed and hence the hydraulic loading rate. To ascertain the significance of HLR and determine potential limiting HLRs for cost neutrality to the other options, the capital cost of VFWs was determined as a function of HLR (Figure 7-4). The critical operational points are determined as the HLR required for the CAPEX of the VFW to match that of each of the AHFW and NSAF. In terms of the NSAF, the critical HLR was 0.37m/d, 0.485m/d and 0.785m/d for the pumped option and 0.35m/d, 0.46m/d and 0.76m/d for the dosing siphon based VFW, for 100PE, 500PE and 2000PE, respectively. In comparison, the critical HLR was much higher when compared to the AHFW with values of 1.35m/d, 1.0m/d and 0.96m/d for the pumped system and 1.1m/d, 0.95m/d and 0.87m/d for the VFW(s) when considering the 100, 500 and 2000PE sites respectively. In summary, due to the high initial CAPEX costs of the NSAF, a VFW provides an economically viable comparison, even for low HLRs, provided the wastewater treatment site to be upgraded has sufficient land available to accommodate VFW construction. As the design for both the VFWs and the AHFW are similar in terms of footprint and construction materials, a comparable CAPEX for both technologies has been observed, indicated by the flat nature of the curve where the line crossed the AHFW benchmark. The CAPEX for AHFW and NSAF are commonly fixed for design based on population equivalent. However, cost models have been constructed to provide an estimate of AHFW and NSAF CAPEX when determined as a function of HLR (Appendix 6).

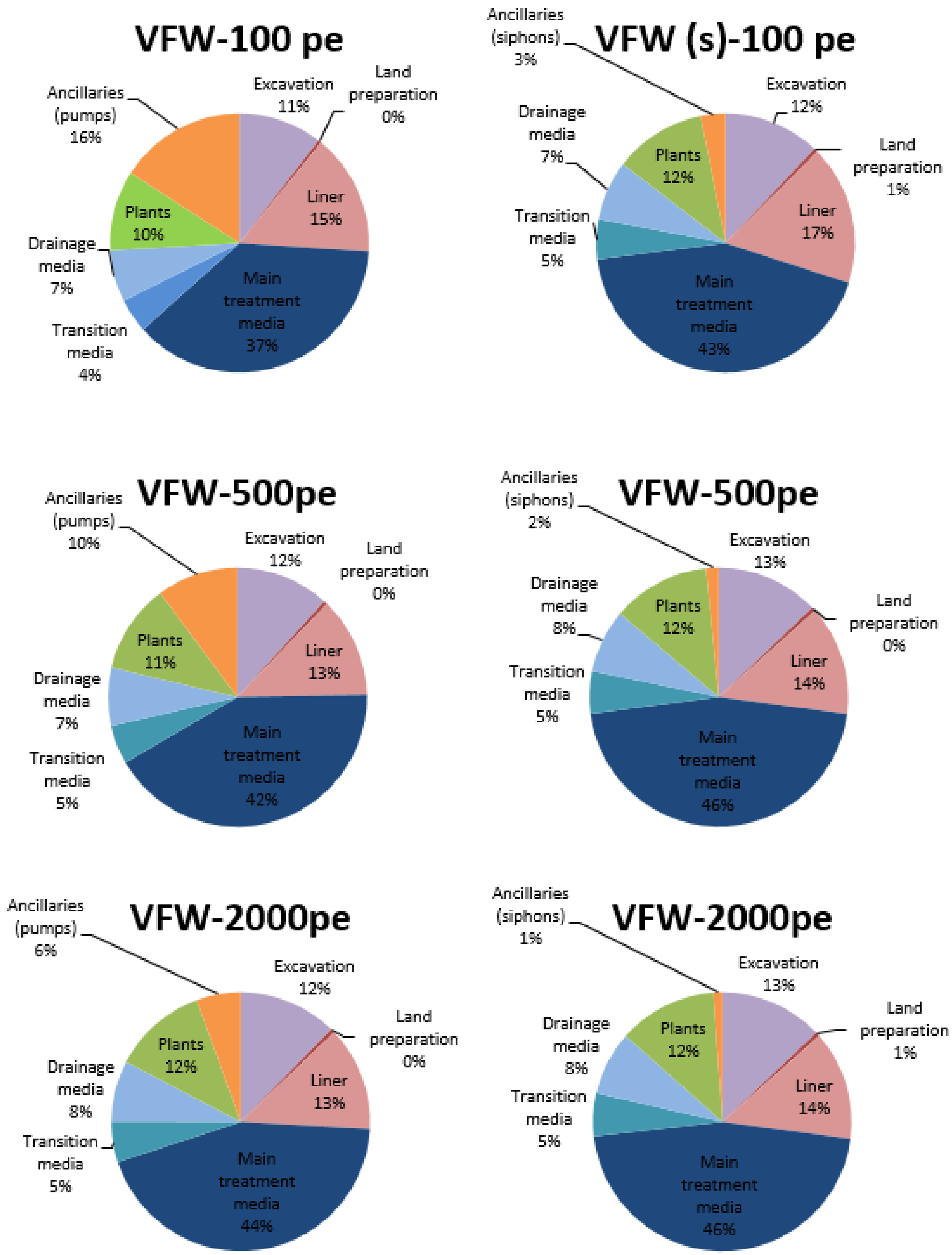

Figure 7-3 Major cost contributors to the CAPEX of VFW's across three different sized designs and comparison between the use of pumps and siphons.

7.5.2 Operational Cost Estimates

Overall OPEX estimations identified the VFW as having the lowest annual operational costs when compared to AHFWs and NSAFs (Table 7-6). This is primarily due to the lower energy requirements associated with the intermittent operation of the feed pumps to the VFW compared (or no energy requirement through the use of siphons) compared to the continual use of air blowers in the AHFW and a combination of continual feed pumps and air blower use and intermittent backwash pumps required for NSAF operation. The greatest OPEX costs associated with 100pe VFW use was determined as the cleaning requirements of the distribution system, in terms of man hours and associated manual labour costs (Figure 7-5). The time requirements for VFW maintenance are conservative estimates based on assumptions made from previous onsite observations and are double the maintenance times specified in the literature. For both the 500pe and 2000pe, the greatest OPEX cost was considered as the refurbishment of media, which occurs every 10 years. For consistency, the cost of wetland media refurbishment for every 10 year period was divided to give an annual estimate. For the AHFW, the greatest operational expenditure was incurred in energy costs for continual operation of the air blow and media refurbishment costs, which was expected due to their slightly larger footprint compared to the VFWs. The major component contributing to the OPEX of the 100pe NSAF was manual labour costs associated with the required weekly maintenance of the systems valves, as specified within the design criteria obtained through a water company. For the 500pe and 2000pe NSAFs, the biggest OPEX contributor was the accumulated energy costs for the feed and backwash pumps and the air blower.

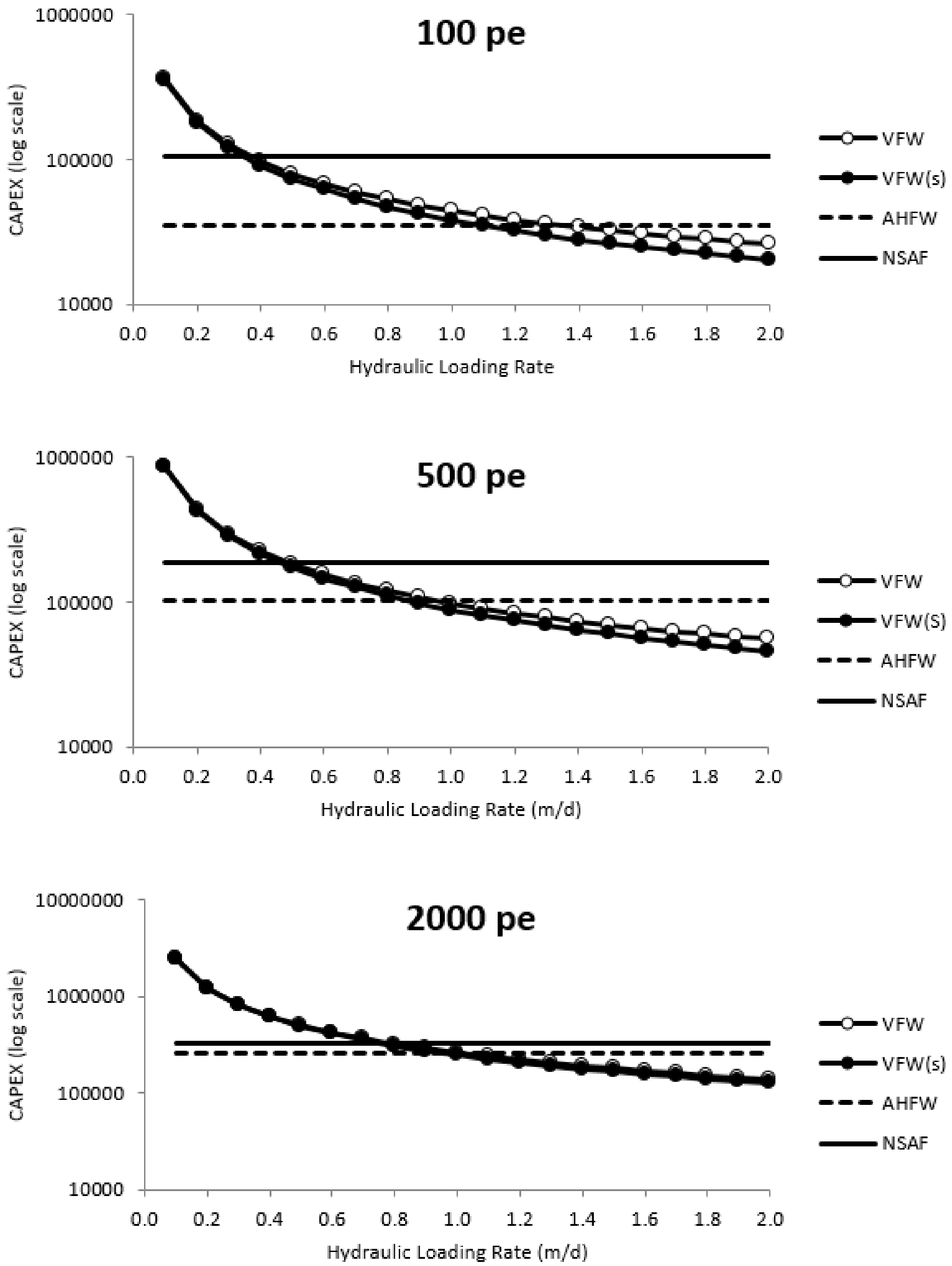

Figure 7-4 Viability of pumped and siphoned VFWs compared to the fixed CAPEX and variable HLR of AHFWs and NSAFs.

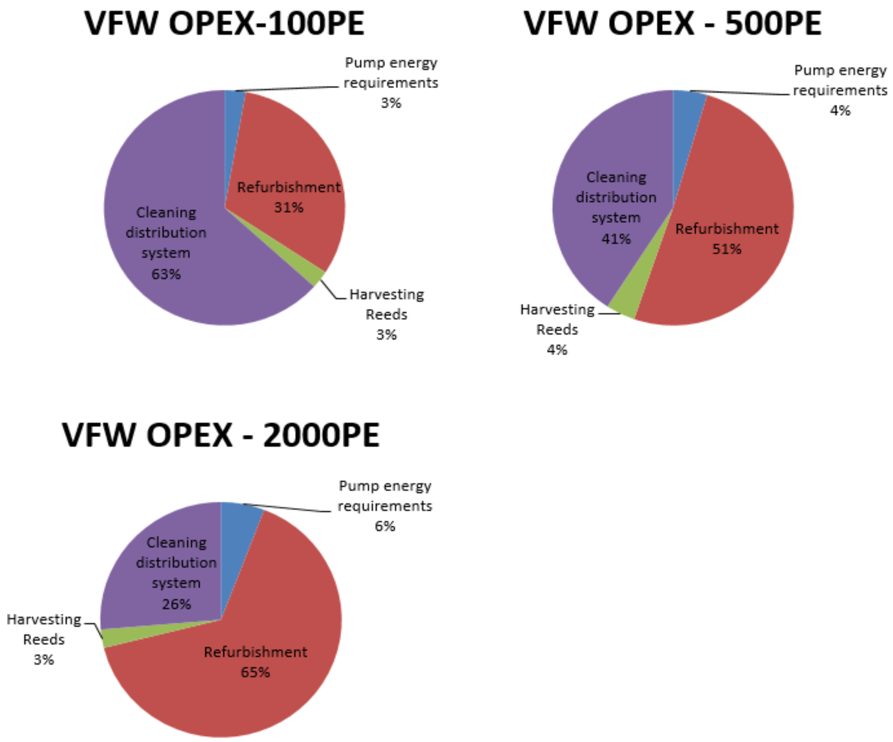

Figure 7-5 Major cost contributors to the OPEX of VFWs across three different sized designs.

Table 7-5 Annual OPEX comparisons for VFWs, AHFWs and NSAFs for designs of population equivalents of 100, 500 and 2000.

	OPEX (£)		
	100pe	500pe	2000pe
VFW	544.05	1687.18	5235.74
VFW(s)	528.75	1610.25	4930.5
AHFW	880.20	2333.98	8567.40
NSAF	1967.16	3059.239	6212.62

7.5.3 Whole Life Costs

Overall WLC analysis determined comparable costs between VFWs and AHFWs across all the three population equivalents included in the study (Figure 7-6). Although the WLC of the AHFW increases with increased population equivalent, notably more in comparison to the VFW, the difference in WLC is accounted for in the 30% variation and is considered insignificant. As the population equivalent increases, the gap between the WLCs of NSAF and the two wetland technologies decrease, suggesting that wetlands may not be considered a viable tertiary treatment option, economically, for wastewater treatment works with population equivalents of over 2000pe. Both the VFWs and AHFWs are considered more economically viable options for inclusion on small works. Vertical flow wetland, AHFW and NSAF data, including typical effluent concentrations achieved by the technologies, were collated and compared to provide the overall viability of the three technologies applied to tertiary application (Table 7-6).

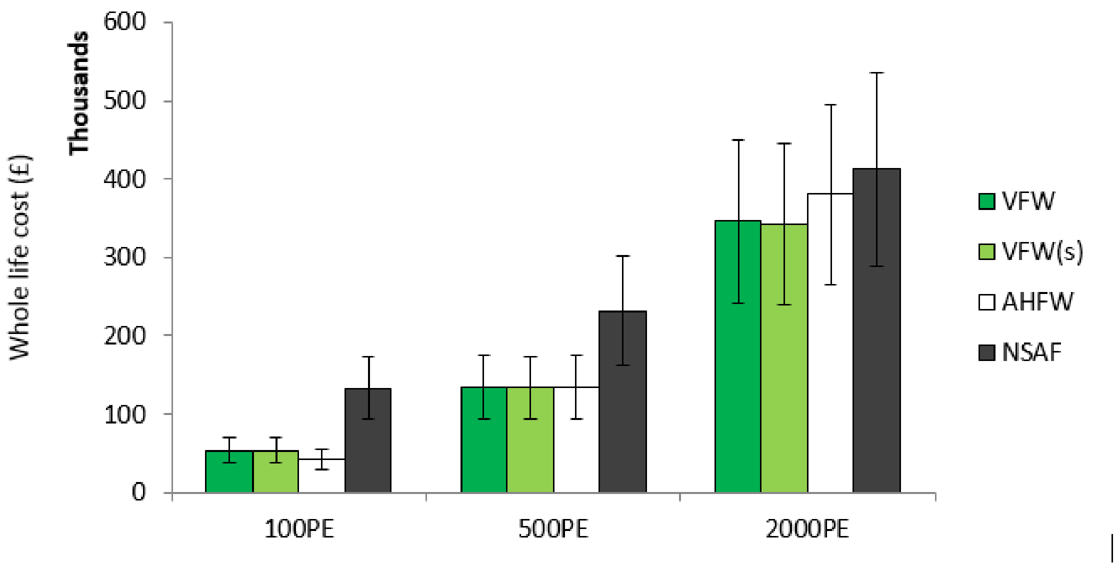

Figure 7-6 WLC estimations for VFWs with pumps (VFW)/siphons (VFW(s)), AHFWs and NSAFs for design capacities of 100, 500 and 2000pe (± 30% variation).

Table 7-6 Overall comparisons in viability for VFW, AHFW and NSAF for wastewater treatment at tertiary application. (VFW and NSAF performance data obtained from the current thesis, AHFW and HFW data obtained from (Butterworth et al., 2013)).

	VFW	NSAF	AHFW	HFW
HLR (m/d) (DWF)	0.26	<0.1	0.3	0.3
Footprint (m^2/pe)	0.57	0.015	0.5	0.5
Energy cost (£/pe)	0.15	2.26	1.80	0
WLC (£/pe)	173	206	190	-
Typical effluent concentrations		Chapters 5, 6		Butterworth et al, 2013
BOD (mg/L)	2.2	-	4.3	7
COD (mg/L)	14.4	45	-	-
NH_4-N (mg/L)	<0.02	1	0.1	8.6
TSS (mg/L)	3.2	16.6	14	21.7
TP (mg/L)	0.3	0.9	-	-

Considerations of reducing the WLC through reduced capital and operational expenditure should include exploration of using alternative main treatment media as sand costs are significantly higher than that of gravel and contribute to approximately 50% of total VFW construction costs. Additionally, consideration should be given to the incorporation of siphons into VFW design as an alternative wetland feeding mechanism to conventional pumping strategies commonly adopted within the UK, thus reducing both the overall VFW CAPEX and OPEX costs.

The impact of not having any energy costs associated with the siphoned VFW reduced the critical HLR required for cost comparability when considering WLC rather than capital cost (Table 7-7). For instance, in the case of the 2000 PE site, the critical HLR for a VFW(s) decreased from 0.95m/d to 0.8m/d when compared to the AHFW and from 0.76 to 0.72m/d when compared to the NSAF. The impact became less significant as the size of the site deceased such that for the 100PE site the critical HLR was the same based on CAPEX or WLC reflecting the increasing importance of CAPEX on WLC as scale decreases through an economy of scale. Comparison to actual HLR indicates that cost appropriateness is likely for the VFWs as the final trials revealed that a HLR of 0.8m/d enabled sustainable operation with no sign of clogging. It is unclear what the limiting HLR of a mature tertiary VFW actually is but it can be anticipated to be higher than the current level suggesting that the analysis provided can be viewed as a conservative assessment. Accordingly, confidence should be afforded towards the economic appropriateness of VFW for tertiary treatment.

Table 7-7 Critical HLR for cost comparability based on CAPEX and WLC

	HLR_{crit}(AHFW) [m/d]		HLR_{crit}(NSAF) [m/d]	
	VFW(S)	VFW	VFW(S)	VFW
WLC				
100	1.1	1.34	0.34	0.37
500	0.78	0.87	0.42	0.44
2000	0.8	0.84	0.72	0.76
CAPEX				
100	1.1	1.35	0.35	0.37
500	0.87	0.96	0.46	0.485
2000	0.95	1.00	0.76	0.785

The analysis conducted in the paper is predicated on an assumption that a HLR of 0.8m/d is possible. However, the initial experimental phases demonstrated a reduced maximum HLR congruent with the requirement for the bed to mature and stabilise as has been documented for VFW used in upstream applications (Molle et al, 2016). Accordingly, consideration of the technology going forward needs to understand how to manage the time period required for bed maturity with restricted HLR expected to be possible during the initial two year period. As such, when first installed only a proportion of the flow should be passed through the VFW, gradually increasing the proportion as the bed matures. This would require a transitional period until the new discharge consents were met or would require temporary treatment to take the other proportion of the flow. This is an ongoing area for consideration and also highlights the need to identify innovative solutions for rapid bed maturation processes for VFWs which could benefit the technology when used across all applications.

7.6 Conclusions

Vertical flow wetlands as a tertiary treatment on small wastewater treatment works provide an economically feasible alternative to AHFW and NSAF technologies. The economic assessment of the VFW has been consistently comparable to that of the AHFW for design population equivalents of 100, 500 and 2000, although VFW's appear to have slightly less overall whole life costs than AHFW for the bigger designed systems. The key cost consideration relates to the capex of the media and hence the design HLR. Analysis revealed that the critical HLRs required for cost comparability are within the range of expected operable HLRs for the technology. The economic assessment was conservatively conducted and as such, confidence can be cautiously afforded towards the economic appropriateness of VFW for tertiary treatment. Key areas for future development that will have a significant impact on the overall economic attractiveness of VFW for tertiary treatment include the ability to use gravel rather than sand, determination of upper operable HLR and innovations that enable rapid bed maturation. Overall, VFW appear a suitable option for tertiary treatment enabling passive delivery of low ammonia discharge concentration

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Chapter 8

Discussion

8 Discussion

The work conducted for this study has demonstrated the use of vertical flow wetlands under tertiary application as an effective approach to achieving the polishing of solids and organics in secondary effluents and enable effective nitrification to be delivered. This was demonstrated through the use of three separate experimental trials, utilising pilot scale VFWs to polish the secondary effluents of a fully operational medium sized wastewater treatment works. Comparisons across all the pilot trials revealed that the VFWs could routinely achieve ammonium-nitrogen effluent concentrations below 2mgNH_4-N/L, with over half of the VFWs achieving below 0.5mgNH_4-N/L. This adheres to the likely future proposed consents of 3-4mgNH_4-N/L and even the sub 1mgNH_4-N/L expected to be introduced on sites discharging to sensitive watercourses. After the initial 20 months of operation, the VFWs were routinely achieving NH_4-N effluent concentrations as low as 0.005mgNH_4-N/L, indicating that the VFWs were encroaching maturity and subsequently approaching their treatment capacity. Additionally, in terms of treatment, the VFWs were able to provide total suspended solids (TSS), chemical oxygen demand (COD) and total phosphorus effluent concentrations of below 16mgTSS/L, 25mgCOD/L and 0.49mgTP/L, respectively, meeting the sites current discharge consents and proving the ability of VFWs to provide overall polishing of secondary effluents to a high standard. When compared to alternative tertiary treatments, the performance of the VFW was similar to that of an artificially aerated horizontal flow wetland (AHFW) in terms of nitrification, but outperformed in terms of solids removal (Butterworth et al., 2013) and exceeded the performance of nitrifying sand filters (SF), nitrifying submerged aerated filter (NSAF) (Chapter 5) and horizontal flow wetlands (HFW) (Butterworth et al., 2013). However, due to the continuous pumped air requirements of the AHFW, the VFW is a more economically viable option for population equivalents of 500 and over (Chapter 7).

During the three experimental phases of the study, the performance of the VFW was not significantly influenced by the applied hydraulic loading rate, the dosing frequency or by the introduction of resting periods, all of which are considered to be key, critical operational parameters for VFW optimisation, thus suggesting that the operating limits had not been reached during the study. It was originally hypothesised that VFWs in tertiary application would be load limited, as opposed to oxygen limited commonly seen in whole and secondary VFW application. The research showed the main limiting factor for successful tertiary VFW operation, particularly during the initial 2 years of operation, to be the hydraulic loading. This subsequently limited the pollutant load available for treatment, therefore agreeing with the research hypothesis. This was shown to be the case as the NH_4-N load removed increased exponentially with increasing influent loading, suggesting capacity for further treatment, had the loads been available. From these findings, it is recommended that the establishment of a new tertiary VFW system requires a start-up period, ideally of at least 2 years, to accommodate a build-up of hydraulic load acceptance, so that optimal operation can be established. This is in agreement with previous studies, whereby a start-up time of between 1-4 years is recommended for wetlands treating raw sewage or primary effluents (Gomez, 2016). However, these recommended start-up times are mainly derived from the time taken to reach optimal treatment performance, as opposed to becoming accepting of hydraulic loads (Chazarenc and Merlin, 2005; Vanier and Dahab, 2001).

The study determined that regular maintenance was essential for the continued operation of the VFWs during the initial 20 months of operation. Excessive hydraulic loadings, and their associated solids loads, appeared to trigger the binding of the media fines on the surface of the VFW, thus preventing the drain down of incoming influents. Once hydraulically overloaded, physical intervention was required to reinitiate flow through the VFW. Hydraulic overloading events had a negative impact of the dissolved oxygen levels within the VFW. Therefore, it is recommended to apply a non-operation recovery period of up to two weeks, as recommended on the French first stage systems, following a clogging event, to allow aerobic degradation and mineralisation of retained internal solids and surface sludge and to allow complete re-oxygenation of the system. To maximise the VFW dissolved oxygen concentration during operation, previous research has recommended using fewer daily doses to feed the VFW with longer rest periods in between feeds, or through applying increased resting periods. The VFWs in this study were sufficiently aerated during normal operation, therefore the benefit of the operational strategies, in terms of re-oxygenation, were not observed during the start-up of the VFWs. With this said, it is postulated that with carefully considered operational strategies, such as applying a low starting HLR with the introduction of small step-wise increments to gradually increase the hydraulic load acceptance, or using alternative set up designs, such as inclusion of additional VFW to incorporate bed resting or load sharing, during the start-up time, hydraulic overloading occurrence will be minimised or avoided completely, reducing the need for intervention.

8.1 References

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Chapter 9

Conclusions

9 Conclusions

The overall findings of this research demonstrated that VFW are a feasible option for tertiary treatment to meet very low ammonia discharge standards on both a performance and economic basis. Specific conclusions that led to this overall conclusion are:

- Vertical flow wetlands (VFWs) are an effective treatment option for ammonia removal at tertiary application. Typical effluent concentrations of 0.3mgNH_4-N/L, 18mgTSS/L and 5.8mgCOD/L for NH_4-N, TSS and COD were achieved during the pilot plant trial showing an overall consistent treatment capacity. The ammonium-nitrogen effluent concentrations achieved throughout the study were below even the tightest proposed future discharge consent of 1mgNH_4-N/L, discharging to sites of specific scientific interest and sensitive watercourses.
- The nitrification performance for VFW in tertiary VFWs has shown to be load limited, as opposed to oxygen limited, as is observed within whole treatment and secondary VFW systems. Residual dissolved oxygen concentrations within the pilot plant VFWs remained above 5mgO_2/L in the influent, 5mgO_2/L internal to the wetland and 4mgO_2/L in the effluent, suggesting nitrification would not be inhibited within these systems based on oxygen availability. Dissolved oxygen concentrations were negatively impacted during periods of hydraulic overloading.
- The operation of VFWs required an initial stabilisation period of between 2 to 3 years to ensure maturation of the system and enable gradual adaptation to increases in hydraulic loadings. This method of operation will prevent the VFW from becoming clogged through hydraulic overloading which will inevitably increase the longevity of the system. This finding is consistent with other VFW studies and means that consideration is required prior to start up.
- Operation of the pilot scale VFWs under different hydraulic loading rates determined that the dosing frequency and application of prolonged resting periods had no significant impact on either the hydraulic stability or treatment efficiency of the VFW system.

- An economic analysis demonstrated the potential economic feasibility of the VFW for tertiary treatment in comparison to existing conventional tertiary treatments. Overall, VFWs have a comparable whole life cost to that of aerated horizontal flow wetlands (AHFW) for use on wastewater treatment sites sub 2000pe, have the potential to operate as energy neutral systems through the use of feeding siphons, and are able to achieve tighter ammonia concentration discharges than those achieved by the AHFW.

Chapter 10

Future Work

10 Future Work

- Findings from the current thesis highlighted an uncertainty as to the maximum hydraulic loading rate that could sustainably be applied to a maturing vertical flow wetland, before the bed becomes subjected to hydraulic overloading. This knowledge could ultimately reduce construction and operational costs associated with VFWs and potentially enhance the feasibility of a VFW compared to existing conventional tertiary wastewater treatments. This research would ideally be conducted on 2-3 year old, maturing VFWs operating under tertiary application and would compare data obtained in relation to hydraulic behaviour and performance potential.
- Following on from the research outlined above, an experimental phase to determine the impact of applying resting periods to pre-clogged VFWs in an attempt to fully restore wetland functionality, could provide insight into the primary clogging mechanisms that occur within VFW at tertiary application. During this study deterioration in solids content within the main treatment media should be assessed using short time intervals to determine the rate at which the beds are able to recover. From these findings a clogging remediation strategy can be implemented for VFW operating under tertiary application.
- In attempt to further reduce VFW initial capital costs and to increase VFW feasibility as an economically comparable tertiary wastewater treatment, the potential of alternative, more cost effective media, such as gravel, for use within the main treatment layer could be assessed. Such studies should include in depth media characterisation to determine the influence of media properties on biofilm attachment and establishment, and adsorption properties. Furthermore, laboratory scale bench top experimental work could be used to determine distribution and settlement patterns of solids to assess the effectiveness of media size on physical pollutant removal.

Appendices

Appendices

Appendix 1: Vertical Flow Wetland CAPEX Calculations

Vertical flow wetland construction requirements and CAPEX calculations for VFWs sized for 100, 500 and 200 population equivalents.

CAPEX – 100PE	**Requirement**	**Unit**	**Associated Cost/Unit**		**CAPEX**	**Unit**
Excavation	85.5	m^3	3.49	£/m^3	298.40	£
Land Preparation	57	m^3	0.21	£/m^3	11.97	£
Liner	87	m^2	4.95	£/m^2	432.63	£
Ancillaries (pumps)					73330.00	£
Main Treatment Media	45.6	tonnes	23.40	£/tonne	1067.04	£
Transition Media	8.55	tonnes	14.40	£/tonne	123.12	£
Drainage Media	12.825	tonnes	14.40	£/tonne	184.68	£
Plants	228		1.25	£/plant	258.00	£
					9732.84	**TOTAL**

CAPEX – 500PE	**Requirement**	**Unit**	**Associated Cost/Unit**		**CAPEX**	**Unit**
Excavation	427.5	m^3	3.49	£/m^3	1491.98	£
Land Preparation	285	m^3	0.21	£/m^3	59.85	£
Liner	323.00	m^2	4.95	£/m^2	1598.85	£
Ancillaries (pumps)					11246.00	£
Main Treatment Media	228	tonnes	23.40	£/tonne	5335.20	£
Transition Media	42.75	tonnes	14.40	£/tonne	615.60	£
Drainage Media	64.125	tonnes	14.40	£/tonne	923.40	£
Plants	1140		1.25	£/plant	1425.00	£
					22695.88	**TOTAL**

CAPEX – 2000PE	**Requirement**	**Unit**	**Associated Cost/Unit**		**CAPEX**	**Unit**
Excavation	1710	m^3	3.49	£/m^3	5967.90	£
Land Preparation	1140	m^3	0.21	£/m^3	239.40	£
Liner	1275.20	m^2	4.95	£/m^2	6312.24	£
Ancillaries (pumps)					15150.00	£
Main Treatment Media	912	tonnes	23.40	£/tonne	21340.80	£
Transition Media	171	tonnes	14.40	£/tonne	2462.40	£
Drainage Media	256.5	tonnes	14.40	£/tonne	3693.60	£
Plants	4560		1.25	£/plant	5700.00	£
					60866.34	**TOTAL**

Appendix 2: Vertical Flow Wetland OPEX Calculations

Vertical flow wetland energy and labour requirements and OPEX calculations for VFWs sized for 100, 500 and 200 population equivalents.

VFW OPEX – 100PE				
Energy Costs	**KW**	**KWh/yr**	**£/unit**	**£/year**
Pump requirements	0.12	180	0.085	15.30
Refurbishment costs	**Frequency**		**£/unit**	**£/year**
Refurbishment	Once/8-12 years		30/m^2	171.00
Manual Labour	**Frequency**	**Time (hours)**	**£/unit**	**£/year**
Reed Harvesting	annually	0.5	26.5	13.25
Cleaning distribution system	fortnightly	0.5	26.5	344.50
			TOTAL	**544.05**

VFW OPEX – 500PE				
Energy Costs	**KW**	**KWh/yr**	**£/unit**	**£/year**
Pump requirements	0.62	905	0.085	76.93
Refurbishment costs	**Frequency**		**£/unit**	**£/year**
Refurbishment	Once/8-12 years		30/m^2	855.00
Manual Labour	**Frequency**	**Time (hours)**	**£/unit**	**£/year**
Reed Harvesting	annually	2.5	26.5	66.25
Cleaning distribution system	fortnightly	1	26.5	689.00
			TOTAL	**1687.18**

VFW OPEX – 2000PE				
Energy Costs	**KW**	**KWh/yr**	**£/unit**	**£/year**
Pump requirements	1.23	3591	0.085	305.24
Refurbishment costs	**Frequency**		**£/unit**	**£/year**
Refurbishment	Once/8-12 years		30/m^2	3420.00
Manual Labour	**Frequency**	**Time (hours)**	**£/unit**	**£/year**
Reed Harvesting	annually	5	26.5	132.50
Cleaning distribution system	fortnightly	2	26.5	1378.00
			TOTAL	**5235.74**

Appendix 3: Aerated Horizontal Flow Wetland OPEX Calculations

Aerated horizontal flow wetland energy and labour requirements and OPEX calculations for AHFWs sized for 100, 500 and 200 population equivalents.

AHFW OPEX – 100PE				
Energy Costs		**KWh/yr**	**£/unit**	**£/year**
Pump requirements		2117	0.085	179.95
Refurbishment costs	**Frequency**		**£/unit**	**£/year**
Refurbishment	Once/8-12 years		30/m^2	210.00
Manual Labour	**Frequency**	**Time (hours)**	**£/unit**	**£/year**
Reed Harvesting	annually	0.5	26.5	13.25
General refurbishment	monthly	0.5	26.5	159.00
			TOTAL	**562.20**

AHFW OPEX – 500PE				
Energy Costs		**KWh/yr**	**£/unit**	**£/year**
Pump requirements		10585	0.085	899.73
Refurbishment costs	**Frequency**		**£/unit**	**£/year**
Refurbishment	Once/8-12 years		30/m^2	1050.00
Manual Labour	**Frequency**	**Time (hours)**	**£/unit**	**£/year**
Reed Harvesting	annually	2.5	26.5	66.25
General refurbishment	monthly	1	26.5	318.00
			TOTAL	**2333.98**

AHFW OPEX – 2000PE				
Energy Costs		**KWh/yr**	**£/unit**	**£/year**
Pump requirements		42340	0.085	3598.90
Refurbishment costs	**Frequency**		**£/unit**	**£/year**
Refurbishment	Once/8-12 years		30/m^2	4200.00
Manual Labour	**Frequency**	**Time (hours)**	**£/unit**	**£/year**
Reed Harvesting	annually	5	26.5	132.50
General refurbishment	monthly	2	26.5	636.00
			TOTAL	**8567.40**

Appendix 4: Nitrifying Submerged Aeration Filter OPEX Calculations

Nitrifying submerged aeration filter energy and labour requirements and OPEX calculations for NSAFs sized for 100, 500 and 200 population equivalents.

NSAF OPEX – 100PE				
Energy Costs	**KW**	**KWh/yr**	**£/unit**	**£/year**
Energy requirements – feed pump	0.12	1051.20	0.085	89.35
Energy requirements – air blowers		2117	0.085	179.95
Energy requirements – backwash pumps	0.12	21.9	0.085	1.86
Manual Labour	**Frequency**	**Time (hours)**	**£/unit**	**£/year**
Flushing of valves	weekly	0.5	26.5	1378.00
Blower maintenance	monthly	1	26.5	318.00
			TOTAL	**1967.16**

NSAF OPEX – 500PE				
Energy Costs	**KW**	**KWh/yr**	**£/unit**	**£/year**
Energy requirements – feed pump	0.62	5431.20	0.085	461.65
Energy requirements – air blowers		10585	0.085	899.73
Energy requirements – backwash pumps	0.12	21.9	0.085	1.86
Manual Labour	**Frequency**	**Time (hours)**	**£/unit**	**£/year**
Flushing of valves	weekly	0.5	26.5	1378.00
Blower maintenance	monthly	1	26.5	318.00
			TOTAL	**3059.24**

NSAF OPEX – 2000PE				
Energy Costs	**KW**	**KWh/yr**	**£/unit**	**£/year**
Energy requirements – feed pump	1.23	10774.80	0.085	915.86
Energy requirements – air blowers		42340	0.085	3598.90
Energy requirements – backwash pumps	0.12	21.9	0.085	1.86
Manual Labour	**Frequency**	**Time (hours)**	**£/unit**	**£/year**
Flushing of valves	weekly	0.5	26.5	1378.00
Blower maintenance	monthly	1	26.5	318.00
			TOTAL	**6212.62**

Appendix 5: Whole Life Cost Calculations

To assess the viability of the simplified whole life cost calculation provided by a sponsoring water company, a full calculation comparison was made when considering a discount rate of 7% over a 40 year period. The following tables show the one off CAPEX plus the annual discounted OPEX for each operational year. The discounted annual OPEX was determined using the following calculation:

$$\frac{\text{Annual OPEX}}{(1+7\%)^{\wedge\text{year of operation}}}$$

This was then compared to the WLC calculated using the simplified calculation: CAPEX + (OPEX *x* 14).

Vertical Flow Wetland

	VFW - 100PE		VFW - 500PE		VFW - 2000PE			VFW(s)- 100PE		VFW(s)- 500PE		VFW(s)- 2000PE	
Year	**CAPEX**	**OPEX**	**CAPEX**	**OPEX**	**CAPEX**	**OPEX**		**CAPEX**	**OPEX**	**CAPEX**	**OPEX**	**CAPEX**	**OPEX**
0	46111.76	544.05	111088.90	1687.18	273447.30	5235.74		40015.76	528.75	101447.90	1610.25	261239.3	4930.50
1	0	508.46	0	1576.80	0	4893.21		0	494.16	0	1504.91	0	4607.94
2	0	475.19	0	1473.65	0	4573.10		0	461.83	0	1406.45	0	4306.49
3	0	444.11	0	1377.24	0	4273.92		0	431.62	0	1314.44	0	4024.76
4	0	415.05	0	1287.14	0	3994.32		0	403.38	0	1228.45	0	3761.45
5	0	387.90	0	1202.94	0	3733.01		0	376.99	0	1148.09	0	3515.38
6	0	362.52	0	1124.24	0	3488.79		0	352.33	0	1072.98	0	3285.40
7	0	338.81	0	1050.69	0	3260.56		0	329.28	0	1002.78	0	3070.47
8	0	316.64	0	981.95	0	3047.25		0	307.74	0	937.18	0	2869.60
9	0	295.93	0	917.71	0	2847.90		0	287.60	0	875.87	0	2681.87
10	0	276.57	0	857.68	0	2661.58		0	268.79	0	818.57	0	2506.42
11	0	258.47	0	801.57	0	2487.46		0	251.21	0	765.02	0	2342.45
12	0	241.56	0	749.13	0	2324.73		0	234.77	0	714.97	0	2189.20
13	0	225.76	0	700.12	0	2172.65		0	219.41	0	668.20	0	2045.98
14	0	210.99	0	654.32	0	2030.51		0	205.06	0	624.48	0	1912.13
15	0	197.19	0	611.51	0	1897.67		0	191.64	0	583.63	0	1787.04
16	0	184.29	0	571.51	0	1773.53		0	179.11	0	545.45	0	1670.13
17	0	172.23	0	534.12	0	1657.50		0	167.39	0	509.76	0	1560.87
18	0	160.96	0	499.18	0	1549.07		0	156.44	0	476.41	0	1458.76
19	0	150.43	0	466.52	0	1447.73		0	146.20	0	445.25	0	1363.32
20	0	140.59	0	436.00	0	1353.01		0	136.64	0	416.12	0	1274.13
21	0	131.40	0	407.48	0	1264.50		0	127.70	0	388.90	0	1190.78

	VFW - 100PE		VFW - 500PE		VFW - 2000PE			VFW(s)- 100PE		VFW(s)- 500PE		VFW(s)- 2000PE	
Year	**CAPEX**	**OPEX**	**CAPEX**	**OPEX**	**CAPEX**	**OPEX**		**CAPEX**	**OPEX**	**CAPEX**	**OPEX**	**CAPEX**	**OPEX**
22	0	122.80	0	380.82	0	1181.78		0	119.35	0	363.45	0	1112.88
23	0	114.77	0	355.91	0	1104.46		0	111.54	0	339.68	0	1040.07
24	0	107.26	0	332.62	0	1032.21		0	104.24	0	317.46	0	972.03
25	0	100.24	0	310.86	0	964.68		0	97.42	0	296.69	0	908.44
26	0	93.68	0	290.52	0	901.57		0	91.05	0	277.28	0	849.01
27	0	87.55	0	271.52	0	842.59		0	85.09	0	259.14	0	793.47
28	0	81.83	0	253.76	0	787.47		0	79.53	0	242.19	0	741.56
29	0	76.47	0	237.15	0	735.95		0	74.32	0	226.34	0	693.04
30	0	71.47	0	221.64	0	687.80		0	69.46	0	211.53	0	647.71
31	0	66.79	0	207.14	0	642.81		0	64.92	0	197.70	0	605.33
32	0	62.42	0	193.59	0	600.75		0	60.67	0	184.76	0	565.73
33	0	58.34	0	180.92	0	561.45		0	56.70	0	172.67	0	528.72
34	0	54.52	0	169.09	0	524.72		0	52.99	0	161.38	0	494.13
35	0	50.96	0	158.03	0	490.39		0	49.52	0	150.82	0	461.81
36	0	47.62	0	147.69	0	458.31		0	46.28	0	140.95	0	431.59
37	0	44.51	0	138.03	0	428.33		0	43.26	0	131.73	0	403.36
38	0	41.60	0	129.00	0	400.31		0	40.43	0	123.11	0	376.97
39	0	38.88	0	120.56	0	374.12		0	37.78	0	115.06	0	352.31
CAPEX		46111.76		111088.90		273447.30			40015.76		101447.90		261239.30
OPEX		7760.83		24067.50		74687.46			7542.58		22970.10		70333.23
WLC		**53872.59**		**135156.40**		**348134.76**			**47558.34**		**124418.00**		**331572.53**
CAPEX + (14*OPEX)		**53728.46**		**134709.42**		**346747.66**			**47418.26**		**123991.40**		**330266.30**

Aerated Horizontal Flow Wetlands and Nitrifying Submerged Aerated Filter

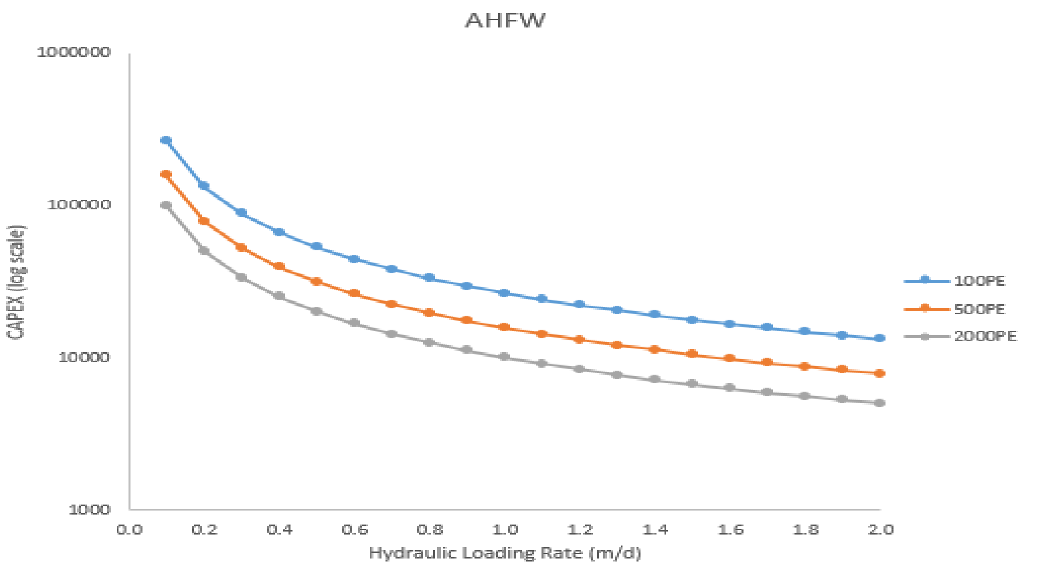
	AHFW- 100PE		**AHFW- 500PE**		**AHFW- 2000PE**			**NSAF- 100PE**		**NSAF- 500PE**		**NSAF- 2000PE**	
Year	**CAPEX**	**OPEX**	**CAPEX**	**OPEX**	**CAPEX**	**OPEX**		**CAPEX**	**OPEX**	**CAPEX**	**OPEX**	**CAPEX**	**OPEX**
0	34581	880.2	102278	2333.98	260274	8567.4		105447	1967.16	189285	3059.24	325409	6212.62
1	0	822.62	0	2181.29	0	8006.92		0	1838.47	0	2859.10	0	5806.19
2	0	768.80	0	2038.59	0	7483.10		0	1718.19	0	2672.06	0	5426.34
3	0	718.51	0	1905.22	0	6993.55		0	1605.79	0	2497.25	0	5071.35
4	0	671.50	0	1780.58	0	6536.03		0	1500.74	0	2333.88	0	4739.58
5	0	627.57	0	1664.10	0	6108.44		0	1402.56	0	2181.20	0	4429.51
6	0	586.51	0	1555.23	0	5708.82		0	1310.80	0	2038.50	0	4139.73
7	0	548.14	0	1453.49	0	5335.35		0	1225.05	0	1905.14	0	3868.91
8	0	512.28	0	1358.40	0	4986.30		0	1144.91	0	1780.51	0	3615.80
9	0	478.77	0	1269.53	0	4660.10		0	1070.00	0	1664.02	0	3379.25
10	0	447.45	0	1186.48	0	4355.23		0	1000.00	0	1555.16	0	3158.18
11	0	418.18	0	1108.86	0	4070.31		0	934.58	0	1453.42	0	2951.57
12	0	390.82	0	1036.32	0	3804.03		0	873.44	0	1358.34	0	2758.48
13	0	365.25	0	968.52	0	3555.17		0	816.30	0	1269.48	0	2578.02
14	0	341.36	0	905.16	0	3322.59		0	762.90	0	1186.43	0	2409.36
15	0	319.02	0	845.94	0	3105.22		0	712.99	0	1108.81	0	2251.74
16	0	298.15	0	790.60	0	2902.07		0	666.35	0	1036.27	0	2104.43
17	0	278.65	0	738.88	0	2712.22		0	622.75	0	968.48	0	1966.76
18	0	260.42	0	690.54	0	2534.78		0	582.01	0	905.12	0	1838.09
19	0	243.38	0	645.36	0	2368.96		0	543.94	0	845.91	0	1717.84
20	0	227.46	0	603.14	0	2213.98		0	508.35	0	790.57	0	1605.46
21	0	212.58	0	563.69	0	2069.14		0	475.09	0	738.85	0	1500.43
22	0	198.67	0	526.81	0	1933.77		0	444.01	0	690.51	0	1402.27

	AHFW- 100PE		AHFW- 500PE		AHFW- 2000PE			NSAF- 100PE		NSAF- 500PE		NSAF- 2000PE	
Year	**CAPEX**	**OPEX**	**CAPEX**	**OPEX**	**CAPEX**	**OPEX**		**CAPEX**	**OPEX**	**CAPEX**	**OPEX**	**CAPEX**	**OPEX**
23	0	185.68	0	492.35	0	1807.27		0	414.97	0	645.34	0	1310.53
24	0	173.53	0	460.14	0	1689.03		0	387.82	0	603.12	0	1224.80
25	0	162.18	0	430.03	0	1578.54		0	362.45	0	563.66	0	1144.67
26	0	151.57	0	401.90	0	1475.27		0	338.74	0	526.79	0	1069.79
27	0	141.65	0	375.61	0	1378.75		0	316.58	0	492.32	0	999.80
28	0	132.38	0	351.04	0	1288.56		0	295.87	0	460.12	0	934.39
29	0	123.72	0	328.07	0	1204.26		0	276.51	0	430.02	0	873.26
30	0	115.63	0	306.61	0	1125.47		0	258.42	0	401.88	0	816.13
31	0	108.06	0	286.55	0	1051.85		0	241.51	0	375.59	0	762.74
32	0	101.00	0	267.80	0	983.03		0	225.71	0	351.02	0	712.84
33	0	94.39	0	250.28	0	918.72		0	210.95	0	328.06	0	666.21
34	0	88.21	0	233.91	0	858.62		0	197.15	0	306.60	0	622.62
35	0	82.44	0	218.61	0	802.45		0	184.25	0	286.54	0	581.89
36	0	77.05	0	204.31	0	749.95		0	172.20	0	267.79	0	543.82
37	0	72.01	0	190.94	0	700.89		0	160.93	0	250.27	0	508.25
38	0	67.30	0	178.45	0	655.04		0	150.40	0	233.90	0	475.00
39	0	62.89	0	166.77	0	612.18		0	140.56	0	218.60	0	443.92
CAPEX		34581.00		102278.00		260274.00			105447.00		189285.00		325409.00
OPEX		12555.99		33294.06		122213.35			28061.40		43639.84		88622.58
WLC		**47136.99**		**135572.06**		**382487.35**			**133508.40**		**232924.84**		**414031.58**
CAPEX + (14*OPEX)		**46903.80**		**134953.72**		**380217.60**			**132987.24**		**232114.36**		**412385.68**

Appendix 6: Cost Models

The cost models for the AHFW and NSAF were estimated by adjusting the relative treatment size per person with differing HLRs. Costs were then estimated based on the relative person equivalent for the technology.

Aerated Horizontal Flow Wetlands



Nitrifying Submerged Aerated Filter